Geotechnical Engineering Report

Dawson's Ridge Density Transfer Subdivision
NW McIntosh Road
Camas, Washington

Prepared for:

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April 7, 2017
Project No. 73197.000

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1.0 INTRODUCTION

1.1 General

This report presents the results of the PBS Engineering and Environmental Inc. (PBS) geotechnical engineering evaluation for the proposed Dawson's Ridge Density Transfer Subdivision Development located along NW McIntosh Road in Camas, Washington. The general site location is shown on the Vicinity Map, Figure 1. The exploration locations in relation to existing and proposed site features are shown on the Site Plan, Figure 2.

1.2 Purpose and Scope

The purpose of PBS' services was to develop geotechnical design and construction recommendations in support of the planned development. This was accomplished by performing the following scope of services.

1.2.1 Literature Review

PBS reviewed various relevant published geologic maps, geologic hazard maps, LiDAR and aerial imagery, topographic maps of the area, and the Clark County GIS system for information regarding geologic conditions. We also reviewed previously completed reports for the project site that were available in our files.

1.2.2 Site Reconnaissance

Portions of the site are located within an area identified by Clark County as "Areas of Potential Instability" (Figure 3, Landslide Hazard and Steep Slopes Maps). A Licensed Engineering Geologist (LEG) from PBS completed a walking reconnaissance of the project area. Mapping was performed by traversing the slope and noting visible geologic features such as outcrops, scarps, cracks, springs, etc., that can be indicators of landslides. These features were photo-documented and are shown on the Site Reconnaissance Map, Figure 5.

1.2.3 Subsurface Exploration

PBS completed 25 test pits at the site (referred to as TP-1 through TP-25). Test pits were excavated to depths ranging from 4.5 feet to 12.5 feet below the existing ground surface (bgs). The test pits were logged and representative soil samples were collected by a PBS engineer. Interpreted test pit logs are included as Figures A1 through A13 in Appendix A, Field Explorations.

1.2.4 Soils Testing

Collected soil samples were transported to our laboratory and classified in general accordance with the Unified Soil Classification, Visual-Manual Procedure. Laboratory tests included natural moisture contents, Atterberg limits, and grain-size analysis (P200) (refer, Appendix B, Laboratory Testing).

1.2.5 Geotechnical Engineering Analysis

Data collected during the subsurface exploration, literature research, site reconnaissance, and laboratory testing was used to develop specific geotechnical design and construction recommendations.

1.2.6 Report Preparation

This Geotechnical Engineering Report summarizes the results of our explorations and analyses, including information relating to the following:

- Exploration logs and site plan with approximate exploration locations
- Site reconnaissance/geologic mapping notes
- Laboratory test results
- Earthwork and grading, cut, and fill recommendations:
  - temporary and permanent slope inclinations
  - building pad preparation
  - utility trench backfill
  - structural fill materials and preparation
  - wet and cold weather conditions consideration
- Shallow foundation design recommendations:
  - minimum embedment
  - allowable bearing pressure
  - estimated settlement
  - sliding coefficient
- Minimum foundation embedment and slope set backs
- Groundwater and drainage considerations
- Seismic design criteria in accordance with the 2015 International Building Code (IBC) with State of Washington amendments
- Recommendations for slope monitoring

1.3 Project Understanding

PBS understands that McIntosh Ridge Holdings, LLC, had previously planned to develop a relatively undeveloped, approximate 30-acre site located on the south side of NW McIntosh Road in Camas, Washington. These development plans included 54 detached, single-family units with 52 of these ranging in lot sizes between 4,500 and 8,000 square feet (sf) and two lots located in the northeastern portion of the property that will be 24,000 sf. Preliminary plans showed these lots located primarily in the center of the butte. An additional 54 townhome units were planned to be constructed, with 11 townhome units plus a recreational building near the center of the complex. These 12 buildings would be constructed on the south side of the property along the butte overlooking the Columbia River and State Route (SR) 14.

Current plans include development of 44 detached, single-family lots ranging in sizes between 10,500 and 18,000 sf. This also includes several private roads and open space areas. No development is currently planned for the south slope areas.

2.0 SITE CONDITIONS

2.1 Geologic Setting

According to the published geologic map of the area (Evarts and O'Connor, 2008), the southwest side of the property along the butte is Pleistocene basaltic andesite of Prune Hill (Qbph), which is overlain by Pleistocene Loess (Qlo) deposits in the flat, upper butte area. The loess is massive, unconsolidated deposits of light-gray eolian silt and fine sand and is mapped only where thick (about 10 to 75 feet) and extensive enough to obscure underlying units. No faults are shown on the US Geological Survey (USGS) Quaternary Faults and Folds Database within 2.5 miles of the site.

2.2 Data Review and Site Reconnaissance

2.2.1 Data Review

Published geologic and geologic hazard maps, LiDAR and aerial imagery, topographic maps, and information available through the Clark County MapsOnline system (http://gis.clark.wa.gov/mapsonline) were reviewed prior to visiting the site. The different maps and imagery were compared to each other for consistencies.

The southwestern portion of the site is within Clark County's designated Landslide Hazard Area and Severe Erosion Hazard Area, which is primarily based on the slope steepness along the southwest side of the property (Figure 3, Landslide Hazard and Steep Slope Maps). Because of these designations, design and development of the project must follow Clark County Code Section 43.430 Geologic Hazard Areas. The discussion and geotechnical recommendations associated with these designations are provided in Sections 3.1 Geotechnical Design Considerations and 3.2 Severe Erosion, Steep Slope, and Potential Slope Instability Hazards.

Fiksdal (1975) used primarily data obtained from low and high altitude aerial photograph review, literature review, and reconnaissance field mapping to identify potential and active slope instabilities. The project area near the butte ridgeline is mapped as an Area 2. An Area 2 has *"potential instability because of underlying geologic conditions and physical characteristics associated with steepness. Geologic and engineering studies are recommended before development."*

Light Detection and Ranging (LiDAR) imagery covering the project area was obtained from Oregon Department of Geology and Mineral Industries (DOGAMI). LiDAR is a remote sensing method that uses light in the form of a pulsed laser to measure ranges (variable distances) to the earth. These light pulses, combined with other data recorded by an airborne system, generate precise three-dimensional information about the shape of the earth and its surface characteristics. Over the past decade, DOGAMI has been collecting and analyzing these data to generate landslide maps and the information is presented through several venues such as the Statewide Landslide Information Database for Oregon (SLIDO).

A pre-historical (> 150 years) colluvium-talus deposit is identified in the SLIDO database along the slope descending between the proposed development and SR 14. The feature was interpreted by DOGAMI through LiDAR imagery and is mapped as a rock fall-type failure that occurred within the basalt bedrock along the steep slopes with the scarp shown as generally following the ridgeline.

2.2.2 Site Reconnaissance

A licensed engineering geologist (LEG) from PBS performed a site reconnaissance on August 6, 2015, to observe the site conditions and, to the extent possible, identify potential landslide-related features within the proximity of the property, primarily along the ridgeline starting near the intersection of NW Brady Road and NW McIntosh Road and walking southeast to the top of the butte. The site reconnaissance was performed by traversing the slopes, noting visible features such as outcrops, scarps, cracks, springs, hummocks, vegetation, and the general geomorphology that may be indicative of ground movement (Figure 5, Site Reconnaissance Map). Due to the heavy vegetative ground cover and steep slopes along the ridgeline, surface cracks and seeps may have been obscured and not apparent during the fieldwork.

The 30-acre site is located atop Prune Hill in Camas, Washington, which is a butte overlooking the Columbia River to the south. The site is bordered by NW McIntosh Ridge on the north, local roads on the west and east, and south-descending ridgeline to State Route 14 on the south.

The Cantera Equestrian Facility, including stables, pens, fields, and training grounds, occupy most of the property. Several residential homes are located on the southern and eastern bounds.

Mature trees, underbrush, and shrubs cover the western and southern sides while most of the upper, flat portion of the site is used for horse corrals and grass fields.

The project site boundary ranges in elevation from 400 feet at the access road entrance near the intersection of NW Brady Road and NW McIntosh Road in the northwest corner to approximately 500 feet above mean sea level (amsl) (based on WGS84 EGM96) at its highest point on the southern side of the butte. The majority of the proposed development sits on the upper, flat butte area and ranges in elevation from 465 to approximately 500 feet amsl. The butte slopes steeply down to the west and south toward the Columbia River with elevations of approximately 120 feet near SR 14 at its base.

The ridgeline has been separated into two areas (West Slope and South Slope) based on the site reconnaissance observations and proposed development (refer, Figure 5).

West Slope: The west slope area is from the property entrance at the intersection of NW Brady Road and NW McIntosh Road along the access road up to TP-13. The topography along the road, which traverses gradually up to the top of the butte, is generally steeper with some exposed rock outcrops and boulders on the east side, and a more gradual slope on the west side that descends toward a stream channel or transitions onto the broader/flatter alluvial terrace.

The site reconnaissance was focused along and adjacent to the access road, observing the conditions and identifying features that could directly affect its stability. The road appears to generally cut across several headscarps of older and/or potentially active landslides. Where fill was placed to cross drainages and level the road, the west sides of these prisms are over-steepened and the pavement has longitudinal cracks, potentially indicating down-slope movement.

The ground surface west and downslope of the road was hummocky and small groups of trees at several locations were tilted. The access road appeared to have been paved relatively recently, possibly in the last 5 to 10 years, making estimations of recent activity along site slopes and associated pavement distress difficult to determine.

South of where the horse trail splits and begins to descend downslope, and the paved road traverses east and upslope (near TP-13), an active landslide has sunken the trail, offset and damaged the fence line, and the trees are tilted downslope. A disconnected and broken drainage line was observed in the upper slope area and the pavement above the drainage head had a linear depression that appeared to be in line with the pipe. The pavement and landscaped area appeared to have recently been improved.

South Slope: The south slope area includes the area from TP-11 to TP-7 as shown on Figure 2. The topography along the ridgeline of the butte, which the horse trail traverses, is relatively flat to the east and descends steeply to the south-southwest toward SR 14. Near TP-25, the ridgeline slopes down and then turns north and upslope for approximately 200 feet along the west side of a drainage channel. The ridgeline then traverses east on the south side of TP-1 along the head of the drainage and away from the development.

Unlike the geomorphology in the west slope area that was incised along a stream channel or transitioned to broader, flatter topography, the south slope area descends steeply, 1H:1V (horizontal to vertical) or steeper, for approximately 400 feet down to SR 14 located at the toe of the slope. Several indicators of potential slope instabilities were observed along the ridgeline, including depressions, transverse cracks, scarps, and debris flow chutes that were generally within 10 feet of the ridgeline. The noted instabilities in the south slope area appear to be high-angle failures with steep headscarps that do not extend a significant distance (< 20 feet) east of the ridgeline.

The ridgeline near TP-10 appears to have a 3- to 5-foot-high scarp that has been excavated into the hill, apparently to create a flat horse trail along the toe of the cut slope. The fence, which is situated along the ridgeline, is tilted, offset, and is being undermined by erosion. A brief observation of the adjacent house did not show readily apparent indications of foundation damage.

An approximately 70-foot-long surface crack above a depressed wedge was observed near TP-9 and likely indicates the top of a debris flow-type landslide. The vegetation within 40 feet of the ridgeline was primarily cleared of trees and consisted of brambles and saplings. Downed and damaged trees were observed further downslope.

Similar features were observed along the remainder of the ridgeline as it traverses south, including the east side of TP-1 and at the head of the drainage near TP-7. The drainage was not accessible due to heavy foliage, fences, and steep terrain during the site reconnaissance. Based on review of site topography developed from LiDAR data, several headscarps and hummocky topography were interpreted.

2.3 Subsurface Conditions

2.3.1 Discussion

Subsurface conditions at the site were explored by excavating 25 test pits (designated TP-1 through TP-25) to depths between 4.5 and 12.5 feet bgs. The test pits were completed on August 11 through 13, 2015, by Dan J. Fischer Excavating, Inc., of Forest Grove, Oregon, using a Deere 310E backhoe equipped with a 24-inch-wide toothed-bucket. Logs summarizing the subsurface conditions encountered in the explorations are presented in Appendix A.

2.3.2 Soil and Bedrock

The soil conditions observed during the subsurface exploration are summarized as follows.

TOPSOIL / GRAVEL FILL:	Grass, shrub, and tree root zones were encountered in the upper 4 inches in the test pits, with localized deeper roots. Test pits TP-11, -14, -15, -16, -22, and -25 encountered fill generally consisting of a silt and gravel-soil mixture. These test pits are located along the access road on the southwest side of the project site and is likely associated with its construction. The thickness of the fill beneath the root zone was approximately 1 to 3 feet.
LOESS DEPOSIT	Below the topsoil and/or fill in the test pits, except for TP-3, TP-6, and TP-25, medium stiff to very stiff SILT or CLAY with variable percentages of fine sand was encountered. The deposit varied from 0 to 12.5 feet thick and increased in thickness over the flat portion of the property. The consistency was medium stiff to very stiff, with DCP correlated N-

values between 6 and 30 blows per foot. The fine-grained materials were non-plastic to highly plastic and moist.

WEATHERED BASALT BEDROCK: Below the topsoil, fill, and/or loess deposits, weathered basalt was encountered in several test pits primarily along the southwestern ridgeline. Table 1 presents the test pits and depths where basalt was observed. The unit generally consisted of red poorly graded GRAVEL (GP-GM) with silt, sand, cobbles, and boulders.

Table 1. Depth to Basalt Bedrock

Test Pit	Depth to Weathered Rock (feet bgs)	Depth of Refusal (feet bgs)
TP-1	7.5	9.5
TP-2	6	10
TP-3	0.5	4.5
TP-6	0.5	7
TP-9	5	9
TP-10	4	8.5
TP-14	5	10
TP-15	n/e[a]	--
TP-16	7	9.5
TP-17	10.5	--
TP-22	8	11.5
TP-25	1.5	7

[a] – rock not encountered during exploration but based on location, should be considered shallow in the immediate vicinity

Figure 2 shows a dashed yellow line that indicates the approximate divide between the test pits where basalt was and was not observed. Deeper excavations should anticipate encountering the basalt throughout the site and the actual depths will vary.

2.3.3 Groundwater

Groundwater was not encountered in any of our explorations except for a perched zone at 12.5 feet bgs in TP-4. We expect seasonal fluctuations of groundwater could occur during extended periods of rainfall or during wet conditions.

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 Geotechnical Design Considerations

Based on our observations and analyses, conventional, shallow spread footing foundations are feasible to support the detached houses. Our current understanding is that a few detached, single-family houses will be constructed adjacent to or within the *Severe Erosion Hazard*, *Slopes > 40 Percent*, and *Areas of Potential Slope Instabilities* zones in the west slope area. Based on local codes, the site reconnaissance, and data review, these structures will require a buffer or setback from the crest of the slope and/or observed landslide scarps. If the project layout changes and houses are placed in the south slope area near the ridgeline, our recommendations will need to be modified to include slope stabilization/mitigation plans prior to building construction.

The subsurface conditions at the site consist of fine-grained soils overlying weathered basalt. The fine-grained silt and clay is generally a thin loess deposit (0.5 feet bgs) near the ridgeline on the west and south sides of the property and, based on TP-1 and TP-13, is at least 12.5 feet bgs approximately 100 feet to the east. The test pit depths where the weathered basalt was encountered indicate the practical excavation refusal depth.

Based on our observations and analyses, conventional, shallow foundations are feasible to support the detached, single-family houses. We recommend that houses on the west side of the yellow dashed bedrock boundary line have their foundations embedded into the underlying basalt bedrock unit and founded entirely on cuts into rock and not on structural fill (refer, Figures 2, 3, and 4).

A grading plan for the project had not been completed when this report was prepared. Subsequently, we have not evaluated the impacts of site grading on the stability of the existing slopes or settlement of the underlying soils. When it is complete, the grading plan should be provided to us for review and evaluation prior to finalizing development plans (pre-design consultation).

3.2 Severe Erosion, Steep Slope, and Potential Slope Instability Hazards

Clark County has mapped portions of the butte ridgeline and slopes on the west and south sides of the site that includes the existing access road within its *Severe Erosion Hazard*, *Slopes > 40 Percent,* and *Areas of Potential Slope Instabilities* zones.

These designations mean development activities must follow specific requirements, including prohibited activities and establishing buffer and setback distances from the slopes, based on the Clark County Code 40.430 Geologic Hazard Areas. The code should be reviewed and additional requirements and submittals will be required by the County for project approval.

3.2.1 West Slope Area

Current plans do not show construction of any structures within the west slope area. The existing access road generally cuts across the headscarps. As a result, PBS recommends not developing the area downslope (west) of the existing access road. Placing fill to widen the road would load the head of the existing landslide deposits and could potentially destabilize the slope. Surface and groundwater flow should be contained, controlled, and diverted away from the road and slopes. If the road were to be considered as a primary access and/or expanded, soil borings, slope stability analyses, and monitoring would need to be performed to further characterize the subsurface conditions and provide additional geotechnical recommendations to conform to the Clark County Code.

3.2.2 South Slope Area

No buildings are proposed within the *Areas of Potential Instability*. Several indicators of slope instability were observed along the ridgeline that will affect the Townhome buffer/setbacks (refer, Figure 3). Four lots are partially located within the *Severe Erosion Hazard* zone (refer, Figure 4). Figures 3 and 4 present the approximate buffer/setback per Clark County Code (dashed orange line) of 50 feet. The types of failures in this area are surficial debris flows and rock falls that tend not to extend significantly beyond the ridgeline. Based on our site reconnaissance, explorations, and literature review, a proposed buffer/setback line (solid red line) has been considered that ranges between 25 and 45 feet from the *Areas of Potential Instability* zone.

According to Clark County Code 40.430.010(B)(3b), the expansion, remodel, reconstruction, or replacement of any structures that will be set back from the geologic hazard area a distance that is greater than or equal to the setback of the original structure, and will not increase the building footprint by more than one thousand (1,000) square feet inside a steep slope hazard area, landslide hazard area, or their buffers, are exempt.

3.3 Seismic Design

3.3.1 Liquefaction

Liquefaction is defined as a decrease of the shear resistance of loose, saturated, cohesionless soil (i.e., sand) or low plasticity silt soils due to the build-up of excess pore-water pressures generated during an earthquake. This results in a temporary transformation of the soil deposit into a viscous fluid. Liquefaction can result in ground settlement, foundation bearing capacity failure, and lateral spreading of ground.

Based on a review of the Liquefaction Susceptibility Map of Clark County, Washington (Palmer, 2004), the site is located in areas of "bedrock" or very low relative liquefaction hazard.

Based on the presence of relatively shallow basalt bedrock at the site and the depth of regional groundwater, the risk of structurally damaging liquefaction settlement at the site is low.

3.3.2 Seismic Design Criteria

The code-based seismic design criteria, in accordance with the 2015 IBC and Washington Amendments, are summarized in Table 2.

Table 2. 2015 IBC Seismic Design Parameters

Parameter	Short Period	1 Second
Maximum Credible Earthquake Spectral Acceleration	S_s = 0.94 g	S_1 = 0.39 g
Site Class	C	
Site Coefficient	F_a = 1.02	F_v = 1.41
Adjusted Spectral Acceleration	S_{MS} = 0.96 g	S_{M1} = 0.55 g
Design Spectral Response Acceleration Parameters	S_{DS} = 0.64 g	S_{D1} = 0.36 g
Design Spectral Peak Ground Acceleration	0.26 g	

g – acceleration due to gravity

3.4 Shallow Foundations

3.4.1 Footing Preparation

PBS recommends that all footing excavations be trimmed neat and footing subgrades carefully prepared. PBS should confirm suitable bearing conditions and evaluate all footing subgrades. Observations should also confirm that loose or soft material, organics, unsuitable fill, and old topsoil zones have been removed from excavations for footings and concrete slabs-on-grade. Localized deepening of footing excavations may be required to penetrate any soft, wet, or deleterious materials.

If construction occurs during wet conditions, we recommend a thin layer of compacted, crushed rock be placed over the footing subgrades to help protect them from disturbance due to foot traffic and the elements. Placement of this rock is the prerogative of the contractor; regardless, the footing subgrade should be in a dense condition prior to pouring concrete.

We recommend a minimum four-inch-thick layer of compacted, crushed rock be placed over the footing subgrades where basalt is exposed to act as a leveling course on top of the excavated rock surface and where fine-grained silt and clay is exposed to protect it from foot traffic.

3.4.2 Footing Embedment Depths

PBS recommends that all footings be founded a minimum of 18 inches below the lowest adjacent grade. The footings should be founded below an imaginary line projecting at a 1H:1V (horizontal to vertical) slope from the base of any adjacent, parallel utility trenches.

Footings for homes located along the "proposed buffer boundary" (orange line, Figure 4), must be embedded a minimum of 2 feet into the weathered basalt bedrock.

3.4.3 Minimum Footing Widths / Design Bearing Pressure

Footings for residential structures should bear on firm native soil and should be sized using a maximum allowable bearing pressure of 2,500 pounds per square foot (psf). Minimum footing widths should be determined based on the applicable local residential building code. The recommended allowable bearing pressure applies to the total of dead plus long-term-live loads. Allowable bearing pressures may be increased by one-third for seismic and wind loads.

Footings for homes whose footings are embedded into the weathered basalt should be sized using a maximum allowable bearing pressure of 3,500 psf. Minimum footing widths should be determined based on the applicable local residential building code.

3.4.4 Foundation Static Settlement

Footings will settle in response to column and wall loads, as well as from the effects of floor live loads. Based on these combined effects and our evaluation of the subsurface conditions, our opinion is that total static settlement will be less than approximately 1 inch.

3.4.5 Lateral Resistance

Lateral loads can be resisted by passive earth pressure on the sides of footings and embedded walls and by friction at the base of the footings. A passive earth pressure of 250 pounds per cubic foot (pcf) may be used for footings confined by native soils and new structural fills. The allowable passive pressure has been reduced by half to account for the large amount of deformation required to mobilize full passive resistance. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent unpaved areas should not be considered when calculating passive resistance. For footings in contact with native granular soils, use a coefficient of friction equal to 0.35 when calculating resistance to sliding. These values do not include a factor of safety.

3.5 Floor Slabs and Modulus of Subgrade Reaction

Satisfactory support for building floor slabs can be obtained from the native silt, clay, or gravel subgrades prepared in accordance with our recommendations presented in the Site Preparation and Wet Weather and Wet Soil Conditions sections of this report. A minimum 6-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade. Imported granular material should be composed of crushed rock or crushed gravel that is relatively well-graded between coarse and fine, contains no deleterious materials, has a maximum particle size of 1 inch, and has less than 5 percent by dry weight passing the US Standard No. 200 Sieve.

For floor slabs supported on subgrades and a base course prepared in accordance with the preceding recommendations, the floor slab may be designed using a modulus of subgrade reaction (k) of 150 pounds per cubic inch (pci).

3.6 Ground Moisture

3.6.1 General

The perimeter ground surface and hard-scaping should be sloped to drain away from all structures and away from adjacent slopes. Gutters should be tight-lined to a suitable discharge and maintained as free-flowing. All crawl spaces should be adequately ventilated and sloped to drain to a suitable, exterior discharge.

3.6.2 Perimeter Footing Drains

Due to the relatively low permeability of site soils and the potential for perched groundwater at the site, we recommend perimeter foundation drains be installed around all proposed structures.

The foundation subdrainage system should include a minimum 4-inch diameter perforated pipe in a drain rock envelope. A non-woven geotextile filter fabric, such as Mirafi 140N or equivalent, should be used to completely wrap the drain rock envelope, separating it from the native soil and footing backfill materials. The invert of the perimeter drain lines should be placed approximately at the bottom of footing elevation. Also, the subdrainage system should be sealed at the ground surface. The perforated subdrainage pipe should be laid to drain by gravity into a non-perforated solid pipe and finally connected to the site drainage stem at a suitable location. Water from downspouts and surface water should be independently collected and routed to a storm sewer or other positive outlet. This water must not be allowed to enter the bearing soils.

3.6.3 Vapor Flow Retarder

A continuous, impervious barrier must be installed over the ground surface in the crawl space and under slabs of all structures. The type of vapor barrier used should be approved by the structural engineer of record and be installed per the manufacturer's recommendations.

4.0 CONSTRUCTION RECOMMENDATIONS

4.1 Site Preparation

Proposed site grading is unknown at this time. However, based on the proposed site plan, mass grading cuts and fills may be up to 5 feet. Stripped vegetation and topsoil should be transported off-site for disposal, or with the owner's approval, stockpiled for re-use in landscaped areas.

4.1.1 Proofrolling

Following site preparation and prior to placing aggregate base or forming and pouring slabs or footings, the exposed subgrade should be evaluated by proofrolling. The subgrade should be proofrolled with a fully-loaded dump truck or similar heavy, rubber-tire construction equipment to identify soft, loose, or unsuitable areas. If evaluation of the subgrades occur during wet conditions, or if proofrolling the subgrades will result in disturbance, they should be evaluated using a steel foundation probe. We recommend that PBS be retained to perform the subgrade verifications.

4.2 Subgrade Protection

4.2.1 Wet Weather and Wet Soil Conditions

Due to the presence of fine-grained soil (i.e. silt and clay) in the near-surface materials within the construction area, construction equipment may have difficulty operating on the near-surface soils when above the optimum moisture required for compaction. Soils that have been disturbed during site preparation activities, or unsuitable areas identified during proofrolling or probing, should be removed to firm ground and replaced with compacted structural fill.

Protection of the subgrade is the responsibility of the contractor. Construction of granular haul roads may help reduce further damage to the exposed native subgrade. The thickness of the granular material for haul roads and staging areas will depend on the amount and type of construction traffic (typically 18 to 24 inches). The actual thickness of haul roads and staging areas should be based on the contractor's approach to site development, and the amount and type of construction traffic. The imported granular material should be placed in lifts no greater than 8 inches in thickness over the prepared, undisturbed subgrade and compacted using a smooth-drum, non-vibratory roller.

During wet conditions, where imported granular material is placed over soft-soil subgrades, we recommend a geotextile be placed between the subgrade and imported granular material. Depending on site conditions, the geotextile should meet WSDOT SS 9-33.2 – Geosynthetic Properties for soil separation or stabilization. The geotextile should be installed in conformance with WSDOT SS 2-12.3 – Construction Geosynthetic (Construction Requirements) and, as applicable, WSDOT SS 2-12.3(2) – Separation or WSDOT SS 2-12.3(3) – Stabilization.

Site earthwork and subgrade preparation should not be completed during freezing conditions. We recommend the earthwork construction at the site be performed during the dry season.

4.2.2 Dry Weather Conditions

Medium to high plasticity clay subgrade soils remaining beneath footings, slabs, or pavements should not be allowed to dry significantly. Once subgrades are approved, the clay soils should be covered within 4 hours of exposure by 4 inches of crushed rock or plastic sheeting during the dry season.

4.3 Excavation

The site soils and the top few feet of the underlying weathered basalt bedrock at the site can be excavated with conventional earthwork equipment. Sloughing and caving should be anticipated. Refusal was encountered during excavation in the weathered basalt bedrock at depths of 4.5 feet to 10.5 feet bgs (refer, Table 1) using the John Deere 310 E backhoe equipped with a 24-inch toothed-bucket. Depending on the depth of site utilities and foundation footings, additional equipment may be required to advance excavations through the potentially harder rock that exists at the site.

For the purposes of this report, rock excavation that may be necessary would apply to subsurface materials that cannot be excavated with a CAT 245 excavator, or equivalent, equipped with rock teeth, and would require systematic drilling or the use of a pneumatic rock hammer. The project schedule and budget should include a contingency for rock excavation and increased backfill volumes. PBS should be retained to review the grading and utility plans when they become available for comparison with encountered field conditions; additional work may be required to better define the impact on the project.

Stockpiled soil should be placed away from the ridgeline and at least 10 feet from the crest of any permanent cut slope. Stockpiled soil should be tracked in using an excavator with slope recommendations consistent with those in Sections 4.4 and 4.5 below.

All excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and State regulations. The contractor is solely responsible for adherence to the OSHA requirements. Trench cuts should stand relatively vertical to a depth of approximately 4 feet bgs, provided no groundwater seepage is present in the trench walls. Open excavation techniques may be used in the clay, silt, silty sand, and sandy silt, provided the excavation is configured in accordance with the OSHA requirements, groundwater seepage is not present, and with the understanding that some sloughing may occur. The trenches should be flattened if sloughing occurs or seepage is present. If shallow groundwater is observed during construction, use of a trench shield or other approved temporary shoring is recommended for cuts that extend below groundwater seepage, or if vertical walls are desired for cuts deeper than 4 feet bgs. If dewatering is used, we recommend that the type and design of the dewatering system be the responsibility of the contractor, who is in the best position to choose systems that fit the overall plan of operation. Perched groundwater was observed at a depth of 12.5 feet bgs in TP-4 excavation, with indications that it may rise seasonally.

4.4 Slopes

Our understanding of the current plans is that the project may include permanent slopes or open excavation. Temporary and permanent cut slopes up to 10 feet high may be inclined at 1.5H:1V and 2H:1V, respectively. Access roads and pavements should be located at least 5 feet from the top of temporary slopes. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face.

4.5 Structural Fill

Single-family houses may be founded in native soil, rock or structural fill and our current understanding is that there may be some fills for site grading. Structural fill, including base rock, should be placed over subgrades that have been prepared in conformance with the Site Preparation and Wet Weather and Wet Soil Conditions sections of this report.

Fill and excavated material placed on slopes steeper than 5H:1V must be keyed/benched into the existing slopes and installed in horizontal lifts. Vertical steps between benches should be approximately 2 feet.

Structural fill should only be installed on subgrades that have been prepared in accordance with the preceding recommendations. Structural fill material should consist of relatively well-graded soil, or an approved rock product that is free of organic material and debris, and contains particles not greater than 4 inches nominal dimension. The suitability of soil for use as compacted structural fill will depend on the gradation and moisture content of the soil when it is placed. As the amount of fines (material finer than the US Standard No. 200 Sieve) increases, soil becomes increasingly sensitive to small changes in moisture content and compaction becomes more difficult to achieve. Soils containing more than about 5 percent fines cannot consistently be compacted into a dense, non-yielding condition when the water content is significantly greater (or significantly less) than optimum.

The silt and clay fraction of soil is moisture sensitive, and during wet weather may become unworkable because of excess moisture content. In order to reduce moisture content, some aerating and drying of native or imported silty/clayey soils may be required. If moisture content of clayey soils cannot be reduced by air drying, it may be necessary to grade the site with granular soils that do not contain more than 5 percent passing the No. 200 Sieve (determined by wet sieve analysis). The imported granular material should be uniformly moisture conditioned to within about 2 percent of the optimum moisture content and compacted in relatively thin lifts using suitable mechanical compaction equipment. We recommend that fine grained fills intended to support the building structures and associated access road sections be placed in horizontal lifts not exceeding about 8 inches in loose thickness and be compacted to at least 92 percent of the maximum dry density as determined by ASTM D 1557 (modified Proctor).

With respect to the current plans, a brief characterization of some of the acceptable materials and our recommendations for their use as structural fill is provided as follows.

4.5.1 Native Soil

Based on our geotechnical exploration, on-site materials are fine-grained soil and weathered basalt bedrock. These may be suitable for mass grading applications. However, due to the difficulty required to dry fine-grained soils to near optimum moisture content, reuse of native silt as structural fill may not be feasible except during dry summer months. Even then, it may require several days of constant mixing in order to achieve the desired moisture content. If used as fill for mass grading, the material should be free of any organic or deleterious material with grain size less than 4 inches in diameter. The material should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM D1557, and shall be placed at a maximum uncompacted thickness of 8 to 12 inches.

4.5.2 Imported Granular Materials

Imported granular material used during periods of wet weather or for haul roads, building pad subgrades, staging areas, etc., should be pit or quarry run rock, crushed rock or crushed gravel, and sand, and should meet the specifications provided in WSDOT SS 9-03.14(2) – Select Borrow. However, the imported granular material should also be fairly well graded between coarse and fine material, and of the fraction passing the US Standard No. 4 Sieve, less than 5 percent by dry weight should pass the US Standard No. 200 Sieve.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 9 inches, and be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

4.5.3 Aggregate Base Course

Imported granular material should be clean, crushed rock or crushed gravel, and sand that is fairly well-graded between coarse and fine. The base aggregate should meet the gradation defined in WSDOT SS 9-03.9(3) – Crushed Surfacing Top Course or Base Course. The base aggregate should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

4.5.4 Foundation Base Aggregate

Imported granular material placed at the base of excavations for spread footings, slabs-on-grade, and other below-grade structures should be clean, crushed rock or crushed gravel, and sand that is fairly well graded between coarse and fine. The granular materials should contain no deleterious materials, have a maximum particle size of 1 inch, and meet WSDOT SS 9-03.12(1)A – Gravel Backfill for Foundations (Class A). The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

4.5.5 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 2 feet above utility lines (i.e., the pipe zone), should consist of well-graded granular material with a maximum particle size of 1 inch and less than 10 percent by weight passing the US Standard No. 200 Sieve, and should meet the standards prescribed by WSDOT SS 9-03.12(3) – Gravel Backfill for Pipe Zone Bedding. The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department.

Within pavement areas or beneath building pads, the remainder of the trench backfill should consist of well-graded granular material with a maximum particle size of 1½ inches, less than 10 percent by weight passing the US Standard No. 200 Sieve, and should meet standards prescribed by WSDOT SS 9-03.19– Bank Run Gravel for Trench Backfill. This material should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department. The upper 2 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D 1557.

Outside of structural improvement areas (e.g., roadway alignments or building pads), trench backfill placed above the pipe zone should consist of excavated material free of wood waste, debris, clods, or rocks greater than 6 inches in diameter and meet WSDOT SS 9-03.14 – Borrow and WSDOT SS 9-03.15 – Native Material for Trench Backfill. This general trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department.

5.0 ADDITIONAL SERVICES RECOMMENDATIONS

If the final development plans remain similar to the provided preliminary layout, several additional work items may need to be completed to conform to Clark County Code. Additional exploration should be anticipated to complete erosion control design, stormwater design, pavement and roadway designs, and any needed slope stability analyses. Final grading plans, detailed erosion control plans, and stormwater designs will need to adhere to Clark County Code requirements and the final plans will need to be reviewed by the geotechnical engineer of record.

PBS should be retained to review the plans and specifications for this project before they are finalized. Such a review allows us to verify that our recommendations and concerns have been adequately addressed in the design.

Satisfactory earthwork performance depends on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. PBS should be retained to observe general excavation, spring and groundwater conditions, stripping, fill placement and compaction, and exposed footing, slab, and pavement subgrades. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations.

Recognition of changed conditions requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated. Unless PBS has the opportunity during construction to confirm the assumptions, interpretations, and analyses, we cannot be held responsible for the applicability of our conclusions and recommendations to subsurface conditions that are different from those anticipated.

In most cases, other services beyond completion of a geotechnical engineering report are necessary or desirable to complete the project. Occasionally, conditions or circumstances arise that require the performance of additional work that was not anticipated when the geotechnical report was written. PBS offers a range of environmental, geological, geotechnical, and construction services to suit the varying needs of our clients.

6.0 LIMITATIONS

This report has been prepared for the exclusive use of the addressee, and their architects and engineers, for aiding in the design and construction of the proposed development and is not to be relied upon by other parties. It is not to be photographed, photocopied, or similarly reproduced, in total or in part, without express written consent of the client and PBS. It is the addressee's responsibility to provide this report to the appropriate design professionals, building officials, and contractors to ensure correct implementation of the recommendations.

The opinions, comments, and conclusions presented in this report are based upon information derived from our literature review, field explorations, laboratory testing, and engineering analyses. It is possible that soil, rock, or groundwater conditions could vary between or beyond the points explored. If soil, rock, or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that PBS is notified immediately so that we may reevaluate the recommendations of this report.

Unanticipated soil and rock conditions and seasonal soil moisture and groundwater variations are commonly encountered and cannot be fully determined by merely taking soil or rock samples from soil borings and test pits. Such variations may result in changes to our recommendations and may require additional funds for expenses to attain a properly constructed project. Therefore, we recommend a contingency fund to accommodate such potential extra costs.

The scope of services for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site.

If there is a substantial lapse of time between the submission of this report and the start of work at the site, if conditions have changed due to natural causes or construction operations at or adjacent to the site, or if the basic project scheme is significantly modified from that assumed, this report should be reviewed to determine the applicability of the conclusions and recommendations presented herein. Land use, site conditions (both on- and off-site), or other factors may change over time and could materially affect our findings. Therefore, this report should not be relied upon after three years from its issue, or in the event that the site conditions change.

7.0 REFERENCES

Clark County. (2015). *Clark County Code, A Codification of the General Ordinances of Clark County, Washington: Section 40.430 Geologic Hazard Areas*. Code Publishing Company, Seattle Washington. Current through Ordinance 2015-08-03, passed August 11, 2015

Evarts, R. C. and O'Connor, J. E. (2008). *Geologic Map of the Camas Quadrangle, Clark County, Washington, and Multnomah County, Oregon. Open-File Report)-06-17*. US Geological Survey, SIM 3017. 1: 24,000.

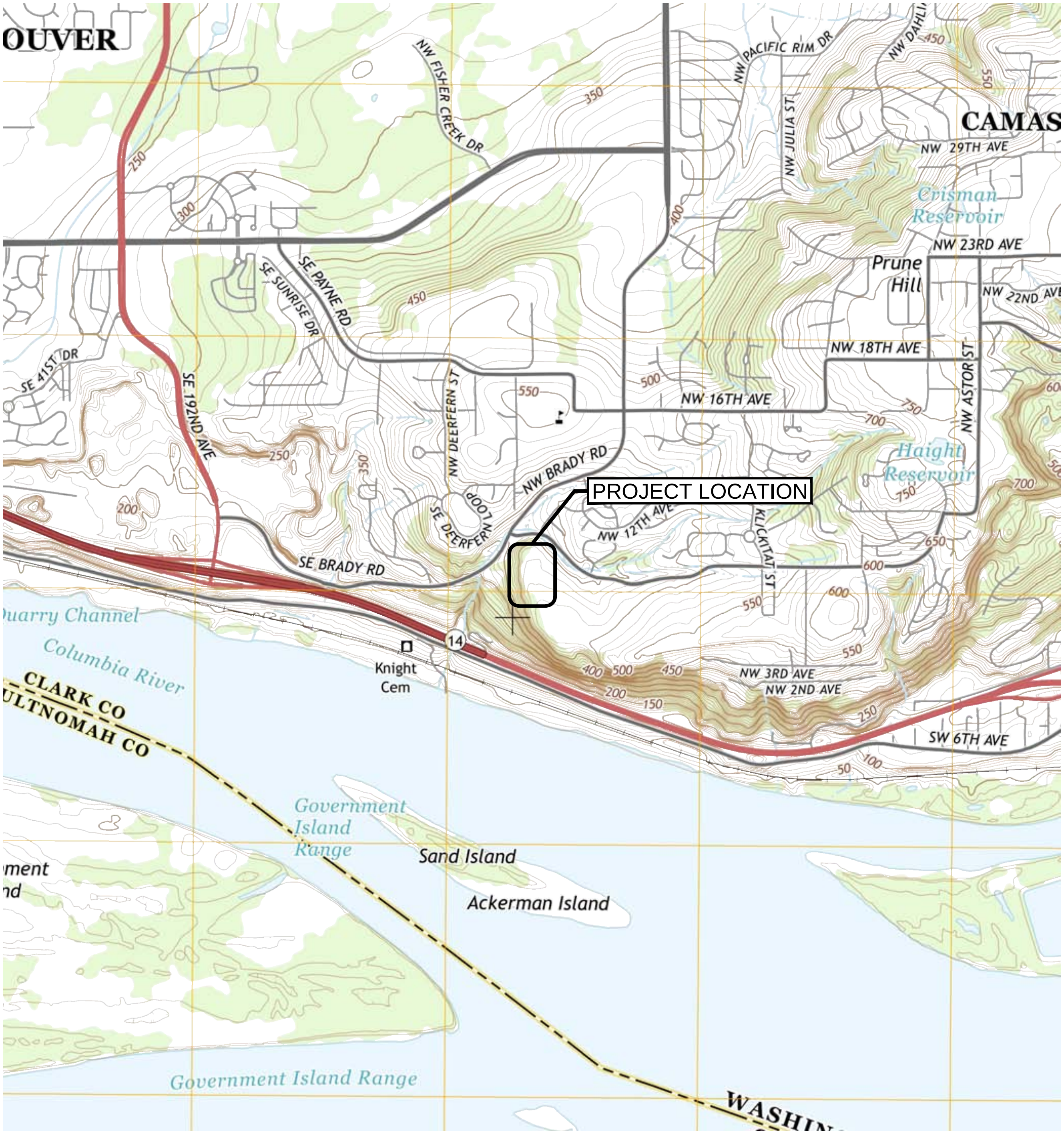
Fiksdal, A. J. (1975). *Slope Stability of Clark County, Washington*. *Open File Report 75-10*. Washington Department of Geology and Earth Resources.

IBC. (2015). *International Building Code*. Country Club Hills, IL: International Code Council, Inc. Washington State Amendments to the International Building Code 2012 Edition.

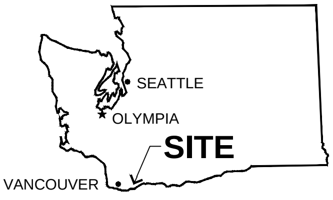
Palmer, S. P, Magsino, S .L, Poelstra J. L, and Niggemann, R. A. (2004). *Liquefaction Susceptibility Map of Clark County, Washington*. Washington Division of Geology and Earth Resources. *Open File Report 2004-20. Liquefaction Susceptibility and Site Class Maps of Washington State, By County*. *Map 6A—Clark County Liquefaction Susceptibility, Sheet 11 of 78*. 1: 100,000

WSDOT SS (2014, amended January 5, 2015). *Standard Specifications for Road, Bridge, and Municipal Construction, M 41-10*. Olympia, WA. Washington State Department of Transportation.

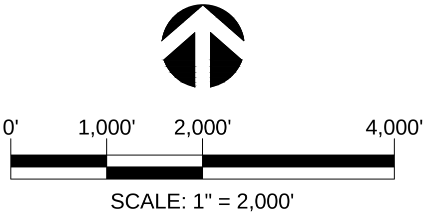
FIGURES



SOURCE: USGS CAMAS WA-OR QUADRANGLE 2013,
PHOTO REVISED 2012.



WASHINGTON



PBS

PROJECT #
73197.000

DATE
APR 2017

VICINITY MAP

NW MCINTOSH ROAD
CAMAS, WASHINGTON

FIGURE
1

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SOURCE: CLARK COUNTY MAPSONLINE 2014 AERIAL IMAGE

LEGEND

▣TP-1 TEST PIT NUMBER AND LOCATION

0' 200' 400' 800'

SCALE: 1" = 400'

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PBS

PROJECT #
73197.000

DATE
APR 2017

SITE PLAN

DAWSON'S RIDGE DEVELOPMENT
NW MCINTOSH ROAD
CAMAS, WASHINGTON

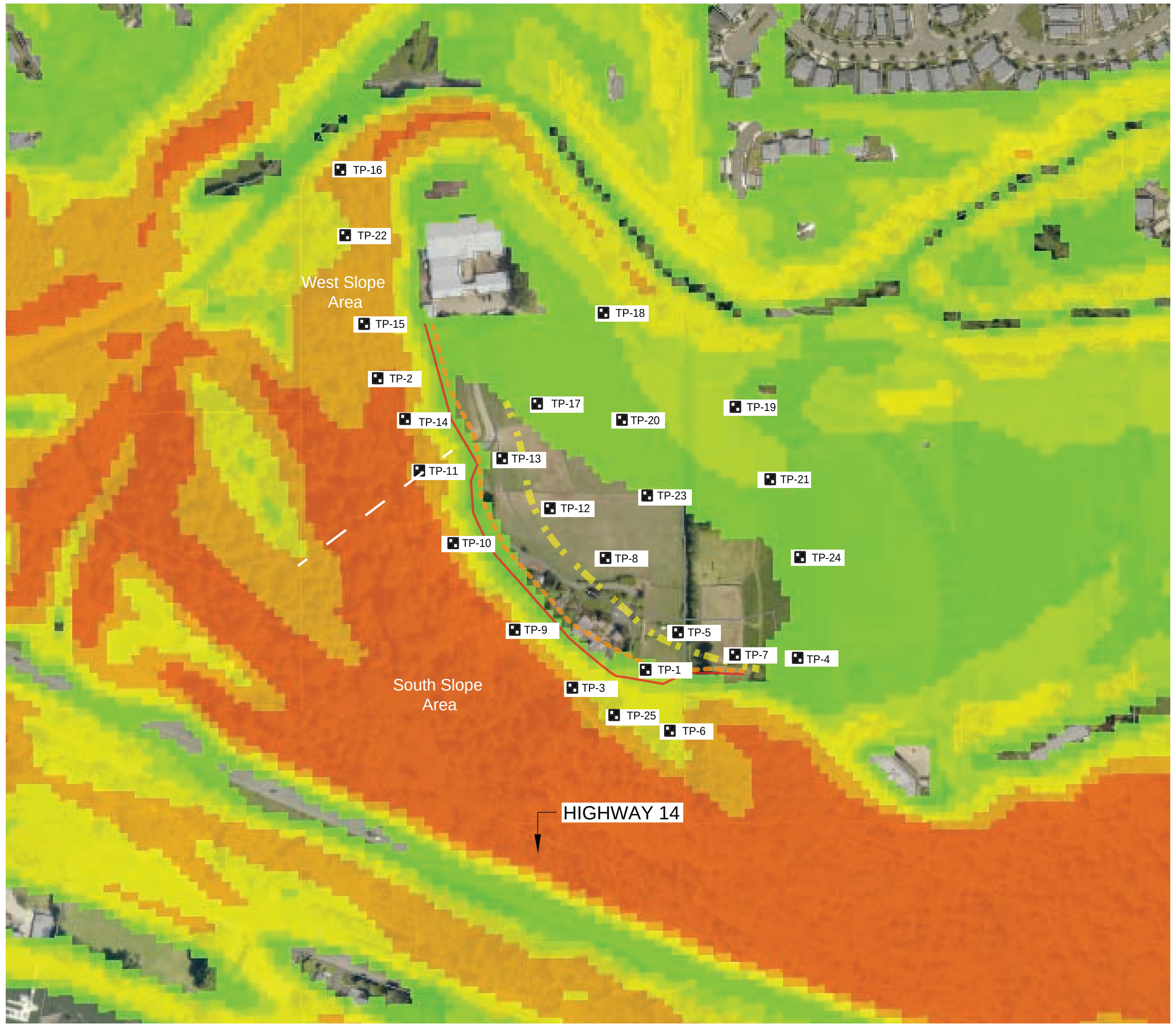
FIGURE
2

LEGEND

Landslide Hazard Areas

▨ Areas of Historic or Active Landslides
▨ Areas of Potential Instability
▨ Areas of Older Landslide Debris
▨ Slopes > 15%
▨ Slopes > 25%
1 TOWNHOME UNIT
- - - BOUNDARY OF BEDROCK OF LESS THAN 10.5 FEET BGS BASED ON TEST PITS
- - - CLARK COUNTY BUFFER BOUNDARY
— PROPOSED BUFFER BOUNDARY
▚ TP-1 TEST PIT NUMBER AND LOCATION

SOURCE: CLARK COUNTY MAPSONLINE, SLOPES AND GEOLOGIC HAZARDS,
clark County, WA. GIS - http://gis.clark.wa.gov
WGS_1984_Web_Mercator_Auxiliary_Sphere



LEGEND

Slopes

☐ less than 5 Percent
🟩 5-10 Percent
🟩 10-15 Percent
🟨 15-25 Percent
🟧 25-40 Percent
🟥 40 - 100 Percent
1 TOWNHOME UNIT
- - - BOUNDARY OF BEDROCK OF LESS THAN 10.5 FEET BGS BASED ON TEST PITS
- - - CLARK COUNTY BUFFER BOUNDARY
— PROPOSED BUFFER BOUNDARY
▚ TP-1 TEST PIT NUMBER AND LOCATION

0' 200' 400' 800'
SCALE: 1" = 400'

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DAWSON'S RIDGE DEVELOPMENT

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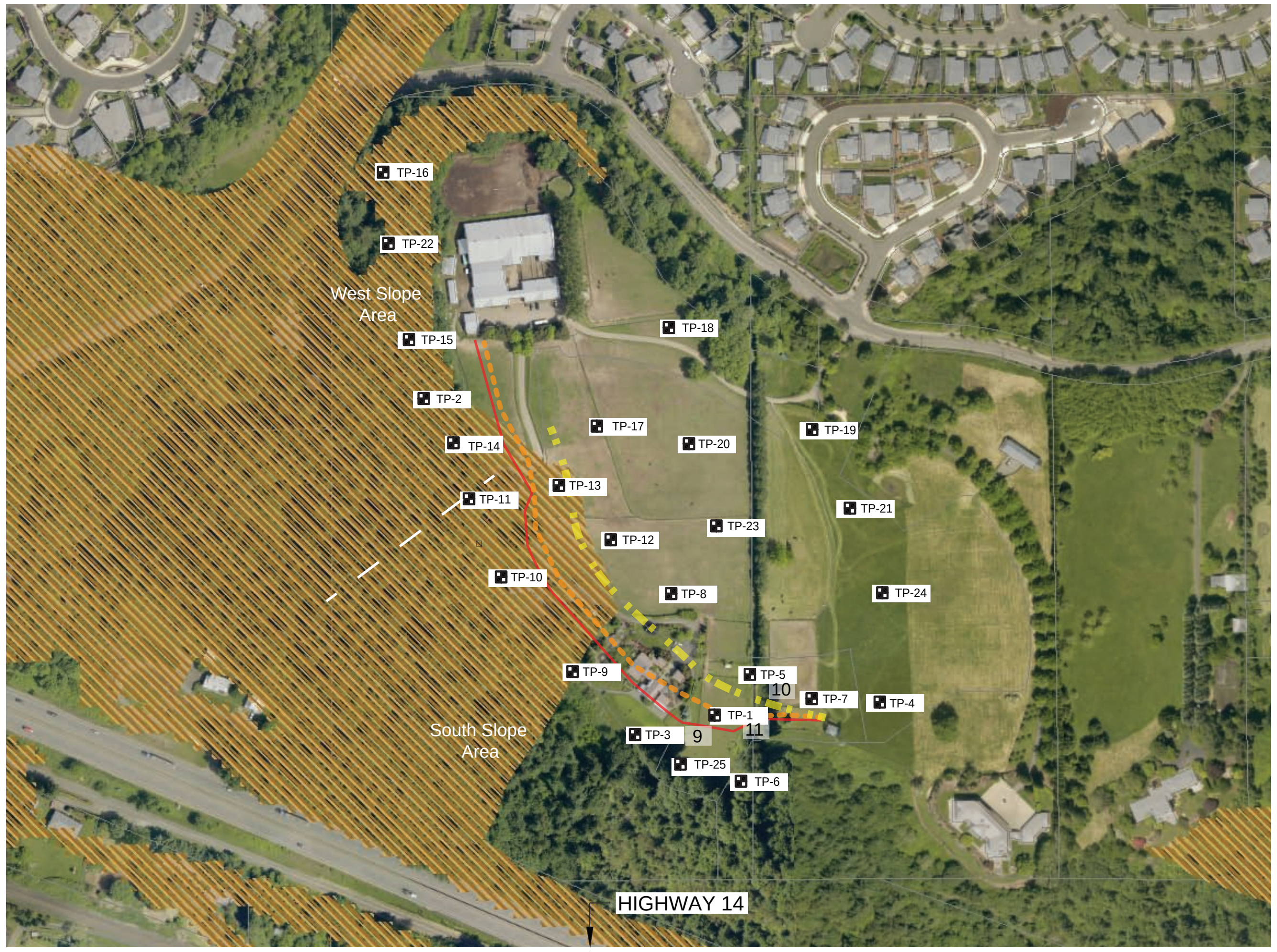
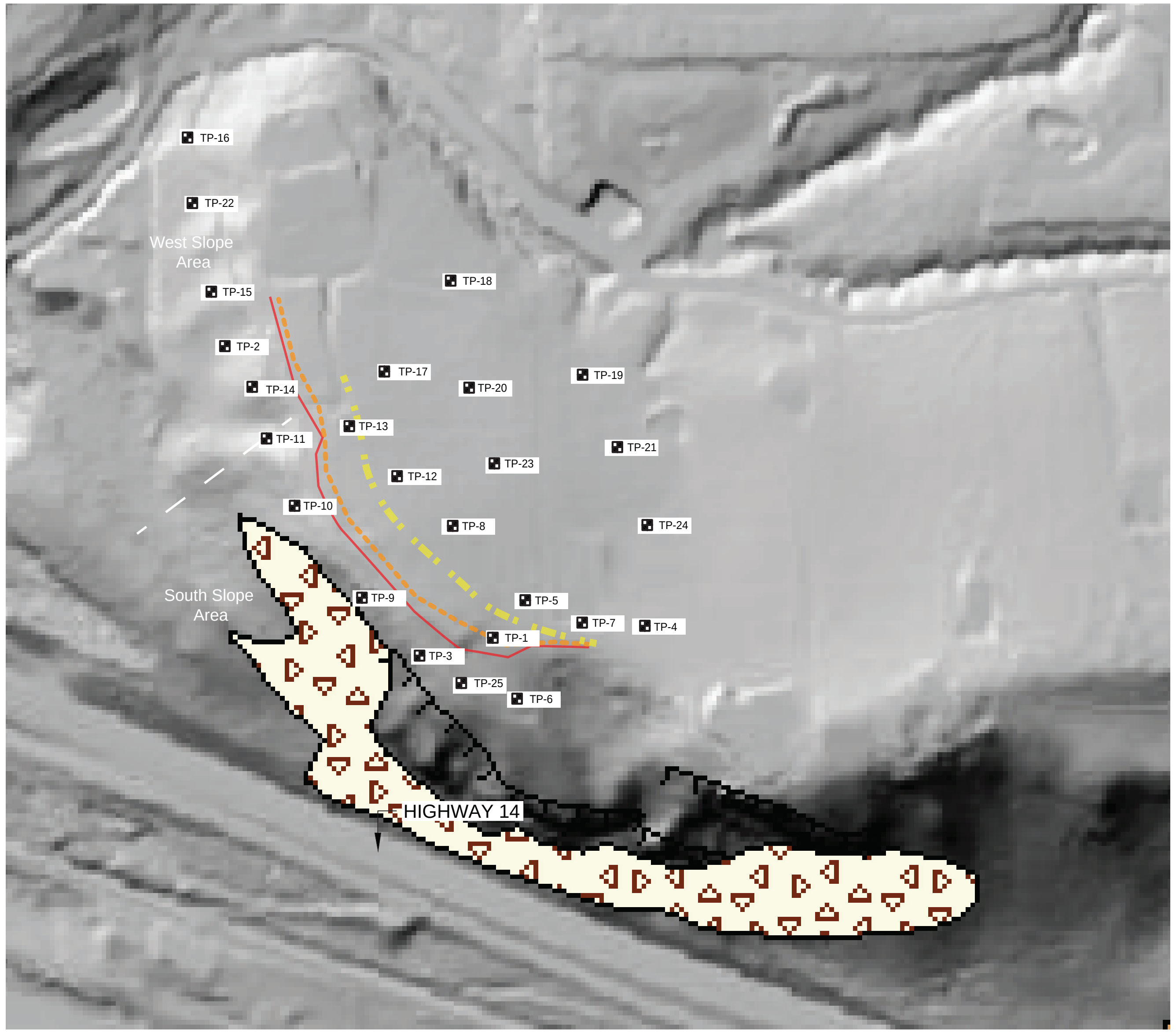
CAMAS, WASHINGTON

LANDSLIDE
HAZARD & STEEP
SLOPES MAPS

PROJECT: 73197.000
DATE: APRIL 2017
FIGURE:
3





LEGEND

Severe Erosion Hazard Area

▨ Severe Erosion Hazard

1 TOWNHOME UNIT

BOUNDARY OF BEDROCK OF LESS THAN 10.5 FEET BGS BASED ON TEST PITS

CLARK COUNTY BUFFER BOUNDARY

PROPOSED BUFFER BOUNDARY

LEGEND

Mapped Landslide Data Inventory

Scarp

Head Scarp

■

Deposits

▨ Talus-Colluvium

🟥 Fan

▨ Landslide

1 TOWNHOME UNIT

BOUNDARY OF BEDROCK OF LESS THAN 10.5 FEET BGS BASED ON TEST PITS

CLARK COUNTY BUFFER BOUNDARY

PROPOSED BUFFER BOUNDARY

0' 200' 400' 800'

SCALE: 1" = 400'

SOURCE: CLARK COUNTY MAPSONLINE, SLOPES AND GEOLOGIC HAZARDS,
clark County, WA. GIS - http://gis.clark.wa.gov
WGS_1984_Web_Mercator_Auxiliary_Sphere

SOURCE: STATEWIDE LANDSLIDE INFORMATION DATABASE FOR OREGON
OREGON DEPARTMENT OF GEOLOGY AND MINERAL INDUSTRIES
http://www.oregongeology.org/sub/slido/

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DAWSON'S RIDGE DEVELOPMENT

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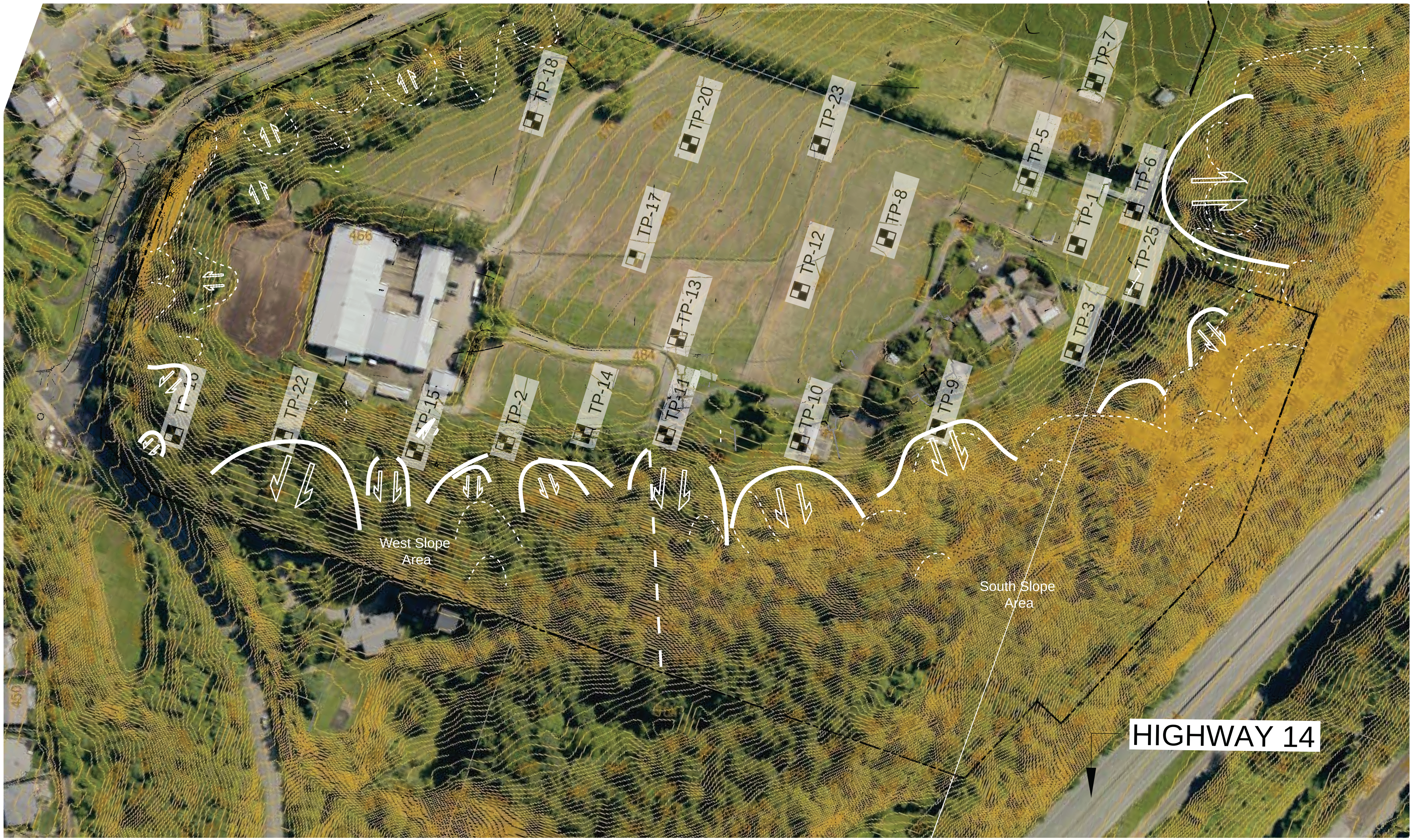
CAMAS, WASHINGTON

EROSION AND SLIDO MAPS

PROJECT: 73197.000
DATE: APRIL 2017
FIGURE: 4



LEGEND

▪TP-1 TEST PIT NUMBER AND LOCATION

1 TOWNHOME UNIT

⇩⇩ DIRECTION OF MOVEMENT

INTERPRETED SLOPE INSTABILITY FROM LIDAR CONTOURS

FIELD OBSERVED SLOPE INSTABILITY

SOURCE: CLARK COUNTY MAPSONLINE, SLOPES AND GEOLOGIC HAZARDS, 2 FOOT LIDAR CONTOURS
clark County, WA. GIS - http://gis.clark.wa.gov
WGS_1984_Web_Mercator_Auxiliary_Sphere

0' 75' 150' 300'

SCALE: 1" = 150'

P

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DAWSON'S RIDGE DEVELOPMENT

NW MCINTOSH ROAD
CAMAS, WASHINGTON

SITE RECON MAP

PROJECT: 73197.000
DATE: APRIL 2017
FIGURE: 5

APPENDIX A

Field Explorations

APPENDIX A – FIELD EXPLORATIONS

A1.0 GENERAL

PBS explored subsurface conditions at the project site by excavating 25 test pits between August 11 and 13, 2015. The approximate locations of the explorations, designated Test Pits TP-1 through TP-25, are shown on Figure 2. The procedures and techniques used to excavate the test pits, collect samples, and other field techniques are described in detail in the following paragraphs. Unless otherwise noted, all soil sampling and classification procedures followed applicable ASTM standards.

A2.0 Test Pits

A2.1 Excavation

Twenty-five test pits were excavated to depths of about 4.5 to 12.5 feet bgs using a Deere 310E backhoe with a 2-foot-wide bucket provided and operated by Dan J. Fischer Excavating, Inc., of Forest Grove, Oregon. The excavations were observed by a PBS geotechnical engineer who maintained a detailed log of the subsurface conditions and materials encountered during the course of the work.

A2.2 Sampling

Disturbed soil samples were collected in the test pit excavations at select depths and lithologic changes. The samples were obtained throughout the excavation from the 2-foot-wide excavation bucket. The disturbed soil samples were examined by the PBS engineer and then sealed in plastic bags for further examination and testing in our laboratory.

At various depths, a Dynamic Cone Penetrometer (DCP) was used to estimate the soil bearing capacity. The DCP consists of two 5/8-inch diameter shafts coupled near the midpoint. The lower shaft contains an anvil and a pointed tip, which is driven into the soil by dropping a 15-pound sliding hammer, contained on the upper shaft, 20-inches onto the anvil. The underlying soil strength is determined by measuring the penetration of the lower shaft into the soil after each hammer drop. The value is recorded in inches per blow and is known as the Dynamic Penetration Index (DPI). The DPI is plotted versus depth and correlated to soil strength parameters such as the relative density / consistency. The values are presented on the test pit logs.

A2.3 Test Pit Logs

The test pit logs show the various types of materials that were encountered in the excavations and the depths where the materials and/or characteristics of these materials changed, although the changes may be gradual. Where material types and descriptions changed between samples, the contacts were interpreted. The types of samples taken during excavation, along with their sample identification number, are shown to the right of the classification of materials. The natural water (moisture) contents are shown further to the right. Measured perched groundwater levels and the dates of the readings are plotted in the column to the right. The groundwater levels are only for the dates shown and will vary from time to time during the year.

A3.0 MATERIAL DESCRIPTION

Initially, soil samples were classified visually in the field. Consistency, color, relative moisture, degree of plasticity, and other distinguishing characteristics of the soil samples were noted. Afterward, the samples were re-examined in the PBS laboratory, various standard classification tests were conducted, and the field classifications were modified where necessary. The terminology used in the soil classifications and other modifiers are defined in Terminology Used to Describe Soil.

Table A-1
Terminology Used to Describe Soil and Rock

Soil Descriptions

Soils exist in mixtures with varying proportions of components. The predominant soil, i.e., greater than 50 percent based on total dry weight, is the primary soil type and is capitalized in our log descriptions (SAND, GRAVEL, SILT, or CLAY). Smaller percentages of other constituents in the soil mixture are indicated by use of modifier words in general accordance with the ASTM D2488-06 Visual-Manual Procedure. "General Accordance" means that certain local and common descriptive practices may have been followed. In accordance with ASTM D2488-06, group symbols (such as GP or CH) are applied on the portion of soil passing the 3-inch (75mm) sieve based on visual examination. The following describes the use of soil names and modifying terms used to describe fine- and coarse-grained soils.

Fine-Grained Soils (50% or greater fines passing 0.075 mm, No. 200 sieve)

The primary soil type, i.e., SILT or CLAY is designated through visual-manual procedures to evaluate soil toughness, dilatency, dry strength, and plasticity. The following outlines the terminology used to describe fine-grained soils, and varies from ASTM D2488 terminology in the use of some common terms.

Primary soil NAME, Symbols, and Adjectives			Plasticity Description	Plasticity Index (PI)
SILT (ML & MH)	**CLAY (CL & CH)**	**ORGANIC SOIL (OL & OH)**		
SILT		Organic SILT	Non-plastic	0 – 3
SILT		Organic SILT	Low plasticity	4 – 10
SILT/Elastic SILT	Lean CLAY	Organic SILT/ Organic CLAY	Medium Plasticity	10 – 20
Elastic SILT	Lean/Fat CLAY	Organic CLAY	High Plasticity	20 – 40
Elastic SILT	Fat CLAY	Organic CLAY	Very Plastic	>40

Modifying terms describing secondary constituents, estimated to 5 percent increments, are applied as follows:

Description	% Composition	
With Sand	% Sand ≥ % Gravel	15% to 25% plus No. 200
With Gravel	% Sand < % Gravel	
Sandy	% Sand ≥ % Gravel	≤30% to 50% plus No. 200
Gravelly	% Sand < % Gravel	

Borderline Symbols, for example CH/MH, are used when soils are not distinctly in one category or when variable soil units contain more than one soil type. **Dual Symbols,** for example CL-ML, are used when two symbols are required in accordance with ASTM D2488.

Soil Consistency terms are applied to fine-grained, plastic soils (i.e., PI ≥ 7). Descriptive terms are based on direct measure or correlation to the Standard Penetration Test N-value as determined by ASTM D1586-84, as follows. SILT soils with low to non-plastic behavior (i.e., PI < 7) may be classified using relative density.

Consistency Term	SPT N-value	Unconfined Compressive Strength	
		tsf	kPa
Very soft	Less than 2	Less than 0.25	Less than 24
Soft	2 – 4	0.25 – 0.5	24 – 48
Medium stiff	5 – 8	0.5 – 1.0	48 – 96
Stiff	9 – 15	1.0 – 2.0	96 – 192
Very stiff	16 – 30	2.0 – 4.0	192 – 383
Hard	Over 30	Over 4.0	Over 383

Soil Descriptions

Coarse - Grained Soils (less than 50% fines)

Coarse-grained soil descriptions, i.e., SAND or GRAVEL, are based on the portion of materials passing a 3-inch (75mm) sieve. Coarse-grained soil group symbols are applied in accordance with ASTM D2488-06 based on the degree of grading, or distribution of grain sizes of the soil. For example, well-graded sand containing a wide range of grain sizes is designated SW; poorly graded gravel, GP, contains high percentages of only certain grain sizes. Terms applied to grain sizes follow.

Material NAME	Particle Diameter	
	Inches	**Millimeters**
SAND (SW or SP)	0.003 – 0.19	0.075 – 4.8
GRAVEL (GW or GP)	0.19 – 3	4.8 – 75
Additional Constituents:		
Cobble	3 – 12	75 – 300
Boulder	12 – 120	300 – 3050

The primary soil type is capitalized, and the fines content in the soil are described as indicated by the following examples. Percentages are based on estimating amounts of fines, sand, and gravel to the nearest 5 percent. Other soil mixtures will have similar descriptive names.

Example: Coarse-Grained Soil Descriptions with Fines

>5% to < 15% fines (Dual Symbols)	≥15% to < 50% fines
Well graded GRAVEL with silt: GW-GM	Silty GRAVEL: GM
Poorly graded SAND with clay: SP-SC	Silty SAND: SM

Additional descriptive terminology applied to coarse-grained soils follow.

Example: Coarse-Grained Soil Descriptions with Other Coarse-Grained Constituents

Coarse-Grained Soil Containing Secondary Constituents	
With sand or with gravel	≥ 15% sand or gravel
With cobbles; with boulders	Any amount of cobbles or boulders.

Cobble and boulder deposits may include a description of the matrix soils, as defined above.

Relative Density terms are applied to granular, non-plastic soils based on direct measure or correlation to the Standard Penetration Test N-value as determined by ASTM D1586-84.

Relative Density Term	SPT N-value
Very loose	0 – 4
Loose	5 – 10
Medium dense	11 – 30
Dense	31 – 50
Very dense	> 50

Rock Descriptions

Scale of Rock Strength

Description	Designation	Unconfined Compressive Strength, psi	Unconfined Compressive Strength, MPa	Field Identification
Extremely weak rock	R0	35 – 150	0.25 – 1	Indented by thumbnail.
Very weak rock	R1	150 – 725	1 – 5	Crumbles under firm blows with point of geology pick; can be peeled by a pocket knife.
Weak rock	R2	725 – 3,500	5 – 25	Can be peeled with a pocket knife; shallow indentation made by firm blow with point of geological hammer.
Medium weak rock	R3	3,500 – 7,000	25 – 50	Cannot by scraped or peeled with a pocket knife; specimen can be fractured with a single firm blow of geological hammer.
Strong rock	R4	7,000 – 15,000	50 – 100	Specimen requires more than one blow with a geological hammer to fracture it.
Very strong rock	R5	15,000 – 36,000	100 – 250	Specimen requires many blows of geological hammer to fracture it.
Extremely strong rock	R6	> 36,000	> 250	Specimen can only be chipped with geological hammer.

Descriptive Terminology for Joint Spacing or Bedding

Descriptive Term	Spacing of Joints	
Very close	< 2 inches	< 50 mm
Close	2 inches – 1 foot	50 mm – 300 mm
Moderately close	1 foot – 3 feet	300 mm – 1 m
Wide	3 feet –10 feet	1 m – 3 m
Very wide	> 10 feet	> 3 m

Descriptive Terminology for Vesicularity

Descriptive Term	Percent voids by volume
Dense	< 1%
Slightly vesicular	1 – 10%
Moderately vesicular	10 – 30%
Highly vesicular	30 – 50%
Scoriaceous	> 50%

Correlation of RQD and Rock Quality

Rock Quality Descriptor	RQD Value
Very poor	0 – 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90

Rock Descriptions

Scale of Rock Weathering

Stage	Description	Quality Distinction
Fresh	Rock is fresh, crystals are bright, few joints may show slight staining as a result of ground water.	No discoloration
Very Slight	Rock is generally fresh, joints are stained, some joints may have thin clay coatings, crystals in broken face show bright.	Discoloration only on major discontinuity surfaces [1]
Slight	Rock is generally fresh, joints are stained and discoloration extends into rock up to 1 inch. Joints may contain clay. In granitoid rocks some feldspar crystals are dull and discolored. Rocks ring under hammer if crystalline.	Discoloration on all discontinuity surfaces and on rock
Moderate	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some are clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.	Decomposition and/or disintegration < 50% of rock [2]
Moderately Severe	All rock, except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick. Rock goes "clunk" when struck.	Decomposition and/or disintegration > 50%, but not complete
Severe	All rock, except quartz, discolored or stained. Rock "fabric" is clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of harder rock usually left, such as corestones in basalt.	
Very Severe	All rock, except quartz, discolored or stained. Rock "fabric" is discernible, but mass effectively reduced to "soil" with only fragments of harder rock remaining.	Decomposition and/or disintegration 100% with structure/fabric intact
Complete	Rock is reduced to "soil." Rock "fabric" is not discernible, or only in small scattered locations. Quartz may be present as dikes or stringers.	Decomposition and/or disintegration 100% with structure/fabric destroyed

NOTES:

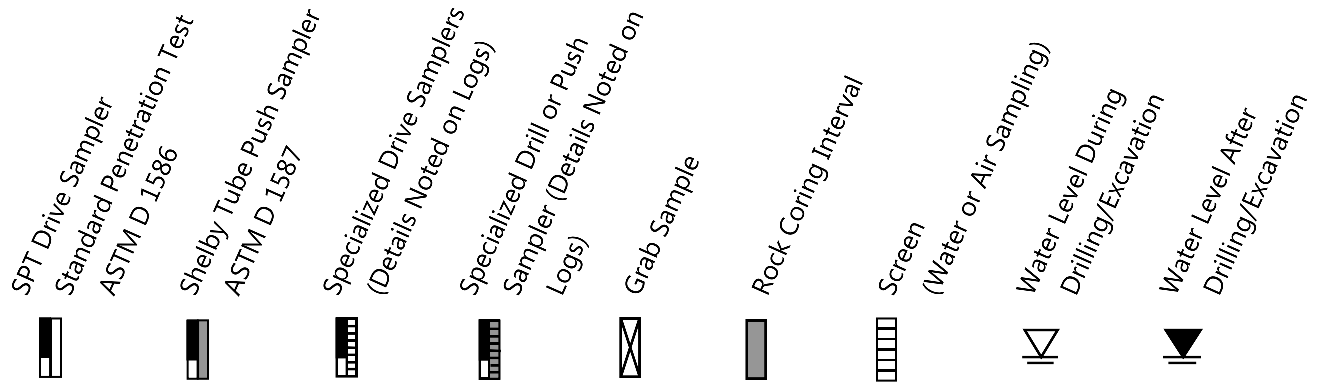
[1] Discontinuities consist of any natural break (joint, fracture or fault) or plane of weakness (shear or gouge zone, bedding plane) in a rock mass

[2] Decomposition refers to chemical alteration of mineral grains; disintegration refers to mechanical breakdown

[3] Stage and description from ASCE Manual No. 56 (1976), quality distinction from Murray (1981)

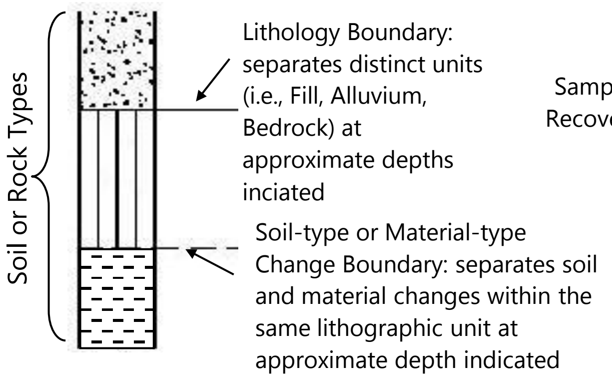
Table A-2
Key To Test Pit and Boring Log Symbols

SAMPLING DESCRIPTIONS

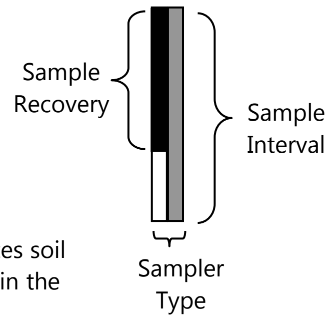


LOG GRAPHICS

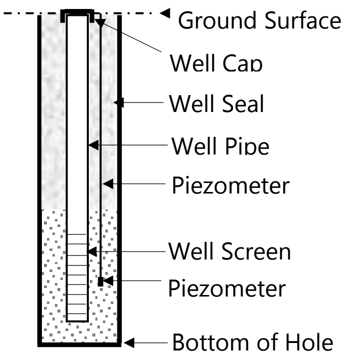
Soil and Rock



Sampling Symbols



Instrumentation Detail



Geotechnical Testing Acronym Explanations

PP	Pocket Penetrometer	HYD	Hydrometer Gradation
TOR	Torvane	SIEV	Sieve Gradation
DCP	Dynamic Cone Penetrometer	DS	Direct Shear
ATT	Atterberg Limits	DD	Dry Density
PL	Plasticity Limit	CBR	California Bearing Ratio
LL	Liquid Limit	RES	Resilient Modulus
PI	Plasticity Index	VS	Vane Shear
P200	Percent Passing US Standard No. 200 Sieve	bgs	Below ground surface
OC	Organic Content	MSL	Mean Sea Level
CON	Consolidation	HCL	Hydrochloric Acid
UC	Unconfined Compressive Strength		

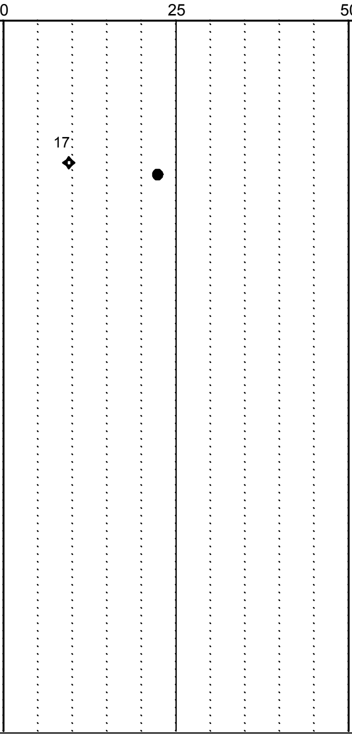
Details of soil and rock classification systems are available on request.

PBS	DAWSON'S RIDGE DEVELOPMENT NW MCINTOSH RD CAMAS, WASHINGTON	**TEST PITS**
	PBS PROJECT NUMBER: 73197.000	TEST PIT LOCATION: (See Site Plan)

DEPTH IN FEET	GRAPHIC LOG	MATERIAL DESCRIPTION NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	◆ DYNAMIC CONE PENETROMETER ☒ STATIC PENETROMETER ● MOISTURE CONTENT %	COMMENTS

TP-1

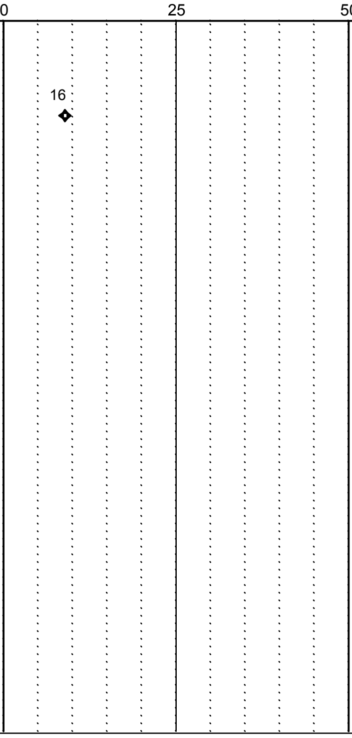
0.0	ROOT ZONE (3 inches)					
	Very stiff brown orange CLAY (CL) with sand; high plasticity; fine sand; moist **LOESS**	0.3	ATT DCP	☒ S-1	17	LL = 43 PL = 22 PI = 21
				☒ S-2		
	Red poorly graded GRAVEL (GP-GM) with silt, sand, cobbles and boulders; non-plastic; fine to coarse sand; coarse, subangular to angular gravel; subangular cobbles; subangular boulders; moist **WEATHERED BASALT BEDROCK**	7.5		☒ S-3		
	Test pit completed at 9.5 feet bgs and backfilled with excavated material. Refusal due to difficult excavation. Groundwater was not observed during excavation.	9.5				



Date Completed: 8/12/2015 Elevation: Final Depth: 9.5 feet bgs Latitude: 45.58239 Longitude: -122.45559

TP-2

0.0	Brown SILT (ML) with sand; non-plastic; fine sand; moist **LOESS**		DCP	☒ S-1	16	
	Red poorly graded GRAVEL (GP-GM) with silt, sand, cobbles and boulders; non-plastic; fine to coarse sand; coarse, subangular to angular gravel; subangular cobbles; subangular boulders; moist **WEATHERED BASALT BEDROCK**	6.0		☒ S-2		
				☒ S-3		
	Test pit completed at 10 feet bgs and backfilled with excavated material. Refusal due to difficult excavation. Groundwater was not observed during excavation.	10.0				



Date Completed: 8/12/2015 Elevation: Final Depth: 10 feet bgs Latitude: 45.58433 Longitude: -122.45753

EXCAVATION METHOD: Backhoe with 24" Bucket
EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: T. Rikli

FIGURE A1

DAWSON'S RIDGE DEVELOPMENT
NW MCINTOSH RD
CAMAS, WASHINGTON

TEST PITS

PBS PROJECT NUMBER:
73197.000

TEST PIT LOCATION:
(See Site Plan)

DEPTH IN FEET	GRAPHIC LOG	MATERIAL DESCRIPTION NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	◆ DYNAMIC CONE PENETROMETER ☒ STATIC PENETROMETER ● MOISTURE CONTENT %	COMMENTS

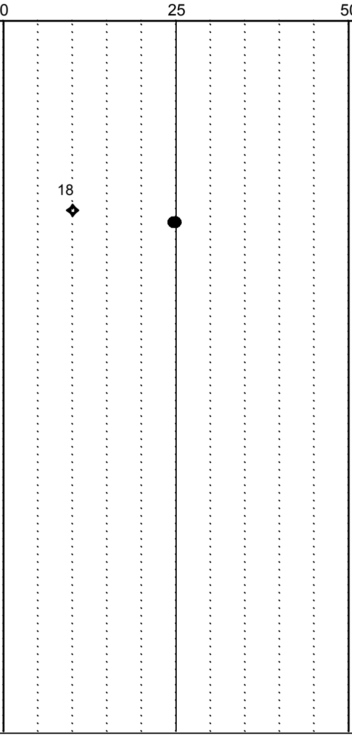
TP-3

Depth	Material Description	Depth	Testing	Sample	Comments
0.0	ROOT ZONE (3 inches)	0.3			
	Red poorly graded GRAVEL (GP-GM) with silt, sand, cobbles and boulders; non-plastic; fine to coarse sand; coarse, subangular to angular gravel; subangular cobbles; subangular boulders; moist **WEATHERED BASALT BEDROCK**			☒ S-1 ☒ S-2	
	Test pit completed at 4.5 feet bgs and backfilled with excavated material. Refusal due to difficult excavation. Groundwater was not observed during excavation.	4.5			

Date Completed: 8/12/2015 Elevation: Final Depth: 4.5 feet bgs Latitude: 45.58224 Longitude: -122.45627

TP-4

Depth	Material Description	Depth	Testing	Sample	Comments
0.0	ROOT ZONE (3 inches)	0.3			
	Very stiff brown orange SILT (ML) with sand; low plasticity; fine sand; moist **LOESS**		P200 DCP	☒ S-1 ☒ S-2 ☒ S-3	P200 = 84% ▼ 08/13/15 Seepage at 12.5 feet
	Test pit completed at 12.5 feet bgs and backfilled with excavated material.	12.5			



Date Completed: 8/13/2015 Elevation: Final Depth: 12.5 feet bgs Latitude: 45.58265 Longitude: -122.45430

EXCAVATION METHOD: Backhoe with 24" Bucket
EXCAVATED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: T. Rikli

FIGURE A3

DAWSON'S RIDGE DEVELOPMENT
NW MCINTOSH RD
CAMAS, WASHINGTON

TEST PITS

PBS PROJECT NUMBER:
73197.000

TEST PIT LOCATION:
(See Site Plan)

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

◆ DYNAMIC CONE PENETROMETER
☒ STATIC PENETROMETER
● MOISTURE CONTENT %

TP-5

DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	◆ DCP (0–25–50)	COMMENTS
0.0	ROOT ZONE (2 inches)	0.2				
	Stiff brown SILT (ML) with sand; low plasticity; fine sand; moist					
4.0			DCP	☒ S-1	13	
6.0	**LOESS**					
8.0				☒ S-2		
12.0				☒ S-3		
12.0	Test pit completed at 12 feet bgs and backfilled with excavated material. Groundwater was not observed during excavation.	12.0				

Date Completed: 8/12/2015 Elevation: Final Depth: 12 feet bgs Latitude: 45.58257 Longitude: -122.45531

TP-6

DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	◆ DCP (0–25–50)	COMMENTS
0.0	ROOT ZONE (3 inches)	0.3				
	Medium dense red poorly graded GRAVEL (GP-GM) with silt, sand, cobbles and boulders; non-plastic; fine to coarse sand; coarse, subangular to angular gravel; subangular cobbles; subangular boulders; moist					
2.0			DCP	☒ S-1	17	
6.0	**WEATHERED BASALT BEDROCK**			☒☒ S-3S-2		
7.0	Test pit completed at 7 feet bgs and backfilled with excavated material. Refusal due to difficult excavation. Groundwater was not observed during excavation.	7.0				

Date Completed: 8/12/2015 Elevation: Final Depth: 7 feet bgs Latitude: 45.58206 Longitude: -122.45541

EXCAVATION METHOD: Backhoe with 24" Bucket
EXCAVATED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: T. Rikli

FIGURE A5

DAWSON'S RIDGE DEVELOPMENT
NW MCINTOSH RD
CAMAS, WASHINGTON

TEST PITS

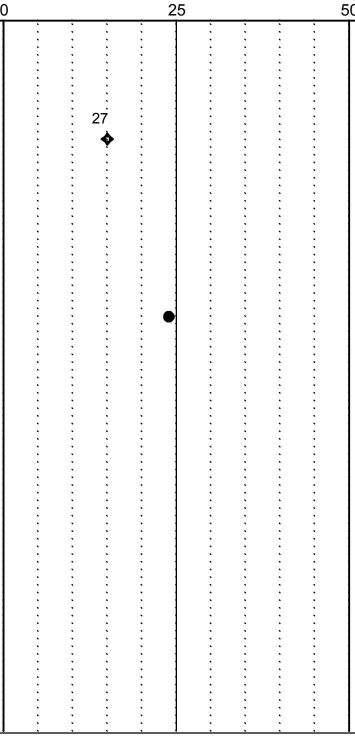
PBS PROJECT NUMBER:
73197.000

TEST PIT LOCATION:
(See Site Plan)

DEPTH IN FEET	GRAPHIC LOG	MATERIAL DESCRIPTION NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	◆ DYNAMIC CONE PENETROMETER ⧗ STATIC PENETROMETER ● MOISTURE CONTENT %	COMMENTS

TP-7

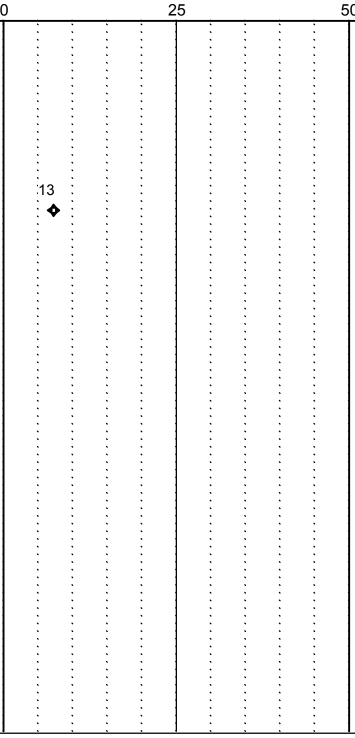
0.0	ROOT ZONE (2 inches)					
	Very stiff brown orange SILT (ML) with sand; low plasticity; fine sand; moist	0.2				
2.0			DCP	☒ S-1	27	
4.0						
6.0	**LOESS**		ATT	☒ S-2	●	LL = 28 PL = 24 PI = 4
8.0						
10.0				☒ S-3		
12.0	Test pit completed at 12 feet bgs and backfilled with excavated material. Groundwater was not observed during excavation.	12.0				
14.0						



Date Completed: 8/12/2015 Elevation: Final Depth: 12 feet bgs Latitude: 45.58316 Longitude: -122.45546

TP-8

0.0	ROOT ZONE (2 inches)					
	Very stiff brown SILT (ML) with sand; low plasticity; fine sand; moist	0.2				
2.0						
4.0			DCP	☒ S-1	13	
6.0	**LOESS**					
8.0				☒ S-2		
10.0						
12.0	Test pit completed at 12 feet bgs and backfilled with excavated material. Groundwater was not observed during excavation.	12.0		☒ S-3		
14.0						



Date Completed: 8/12/2015 Elevation: Final Depth: 12 feet bgs Latitude: 45.58320 Longitude: -122.45598

EXCAVATION METHOD: Backhoe with 24" Bucket
EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: T. Rikli

FIGURE A7

DAWSON'S RIDGE DEVELOPMENT
NW MCINTOSH RD
CAMAS, WASHINGTON

TEST PITS

PBS PROJECT NUMBER:
73197.000

TEST PIT LOCATION:
(See Site Plan)

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

◆ DYNAMIC CONE PENETROMETER
☒ STATIC PENETROMETER
● MOISTURE CONTENT %

TP-9

DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	◆ DCP (0 – 25 – 50)	COMMENTS
0.0	ROOT ZONE (3 inches)	0.3				
	Very stiff brown orange SILT (ML) with sand; low plasticity; fine sand; moist **LOESS**		DCP	☒ S-1	25	
5.0	Red poorly graded GRAVEL (GP-GM) with silt, sand, cobbles and boulders; non-plastic; fine to coarse sand; coarse, subangular to angular gravel; subangular cobbles; subangular boulders; moist **WEATHERED BASALT BEDROCK**	5.0		☒ S-2 ☒ S-3		
9.0	Test pit completed at 9 feet bgs and backfilled with excavated material. Refusal due to difficult excavation. Groundwater was not observed during excavation.	9.0				

Date Completed: 8/12/2015 Elevation: Final Depth: 9 feet bgs Latitude: 45.58274 Longitude: -122.45671

TP-10

DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	◆ DCP (0 – 25 – 50)	COMMENTS
0.0	ROOT ZONE (3 inches)	0.3				
	Stiff brown orange sandy SILT (ML); low plasticity; fine sand; moist **LOESS**		DCP	☒ S-1	10	
4.0	Red poorly graded GRAVEL (GP-GM) with silt, sand, cobbles and boulders; non-plastic; fine to coarse sand; coarse, subangular to angular gravel; subangular cobbles; subangular boulders; moist **WEATHERED BASALT BEDROCK**	4.0		☒ S-2 ☒ S-3		
8.5	Test pit completed at 8.5 feet bgs and backfilled with excavated material. Refusal due to difficult excavation. Groundwater was not observed during excavation.	8.5				

Date Completed: 8/12/2015 Elevation: Final Depth: 8.5 feet bgs Latitude: 45.58359 Longitude: -122.45752

EXCAVATION METHOD: Backhoe with 24" Bucket
EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: T. Rikli

FIGURE A9
Page 5 of 13

DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	◆ DYNAMIC CONE PENETROMETER ⊠ STATIC PENETROMETER ● MOISTURE CONTENT % (0 – 25 – 50)	COMMENTS

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

TP-11

DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	DCP / MOISTURE	COMMENTS
0.0	ROOT ZONE (3 inches)	0.3				
	Brown SILT (ML) with gravel; low plasticity; fine, rounded gravel; moist **FILL**	3.0				
	ROOT ZONE (6 inches)	3.5				
4.0	Brown SILT (ML) with sand; low plasticity; fine sand; moist **LOESS**			☒ S-1		
7.0				☒ S-2		
11.0				☒ S-3		
11.5	Test pit completed at 11.5 feet bgs and backfilled with excavated material. Groundwater was not observed during excavation.	11.5				

Date Completed: 8/12/2015 Elevation: Final Depth: 11.5 feet bgs Latitude: 45.58385 Longitude: -122.45770

TP-12

DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	DCP / MOISTURE	COMMENTS
0.0	ROOT ZONE (2 inches)	0.2				
	Very stiff brown orange SILT (ML) with sand; low plasticity; fine sand; moist **LOESS**					
3.0			DCP	☒ S-1	◆ 18	
7.0				☒ S-2		
11.0	becomes brown red			☒ S-3		
11.5	Test pit completed at 11.5 feet bgs and backfilled with excavated material. Groundwater was not observed during excavation.	11.5				

Date Completed: 8/12/2015 Elevation: Final Depth: 11.5 feet bgs Latitude: 45.58347 Longitude: -122.45637

EXCAVATION METHOD: Backhoe with 24" Bucket
EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: T. Rikli

PBS	DAWSON'S RIDGE DEVELOPMENT NW MCINTOSH RD CAMAS, WASHINGTON	**TEST PITS**
	PBS PROJECT NUMBER: 73197.000	TEST PIT LOCATION: (See Site Plan)

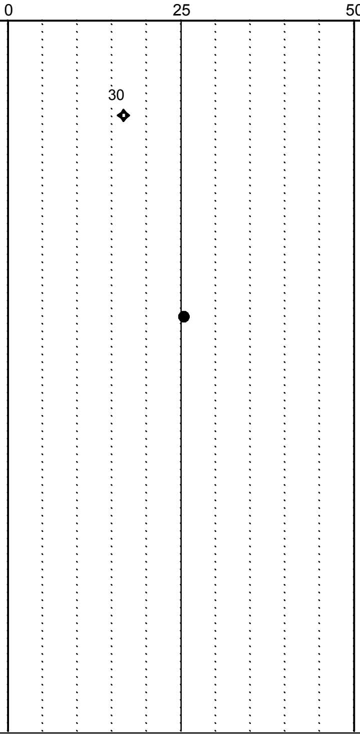
MATERIAL DESCRIPTION

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

◆ DYNAMIC CONE PENETROMETER
☒ STATIC PENETROMETER
● MOISTURE CONTENT %

TP-13

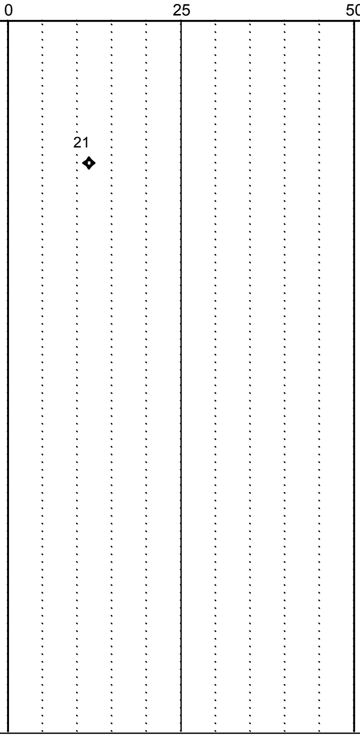
DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	COMMENTS
0.0	ROOT ZONE (2 inches)	0.2			
	Very stiff brown orange SILT (ML) with sand; non-plastic; fine sand; moist **LOESS**				
2.0			DCP	☒ S-1	
6.0			ATT	☒ S-2	LL = 26 PL = 24 PI = 2
10.0				☒ S-3	
	Red poorly graded GRAVEL (GP-GM) with silt, sand, cobbles and boulders; non-plastic; fine to coarse sand; coarse, subangular to angular gravel; subangular cobbles; subangular boulders; moist	11.0			
	Test pit completed at 11.5 feet bgs and backfilled with excavated material. Refusal due	11.5			



Date Completed: 8/12/2015 Elevation: Final Depth: 11.5 feet bgs Latitude: 45.58388 Longitude: -122.45693

TP-14

DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	COMMENTS
0.0	ROOT ZONE (3 inches)	0.3			
	Brown gravelly SILT (ML); non-plastic; coarse, rounded gravel; moist **FILL**	1.5			
	Very stiff brown orange SILT (ML) with sand; low plasticity; fine sand; moist **LOESS**		DCP	☒ S-1	
	Red poorly graded GRAVEL (GP-GM) with silt, sand, cobbles and boulders; non-plastic; fine to coarse sand; coarse, subangular to angular gravel; subangular cobbles; subangular boulders; moist **WEATHERED BASALT BEDROCK**	5.0		☒ S-2	
	Red poorly graded SAND (SP) with gravel; fine sand; fine, rounded gravel; moist	9.0		☒ S-3	
10.0	Test pit completed at 10 feet bgs and backfilled with excavated material. Refusal due to difficult excavation. Groundwater was not observed during excavation.	10.0			



Date Completed: 8/12/2015 Elevation: Final Depth: 10 feet bgs Latitude: 45.58397 Longitude: -122.45731

EXCAVATION METHOD: Backhoe with 24" Bucket
EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: T. Rikli

FIGURE A13

PBS	DAWSON'S RIDGE DEVELOPMENT NW MCINTOSH RD CAMAS, WASHINGTON	**TEST PITS**
	PBS PROJECT NUMBER: 73197.000	TEST PIT LOCATION: (See Site Plan)

DEPTH IN FEET	GRAPHIC LOG	MATERIAL DESCRIPTION NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	◆ DYNAMIC CONE PENETROMETER ☒ STATIC PENETROMETER ● MOISTURE CONTENT % 0 25 50	COMMENTS

TP-15

DEPTH IN FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	DCP	COMMENTS
0.0		ROOT ZONE (3 inches)	0.3				
		Stiff brown orange SILT (ML) with sand; low plasticity; fine sand; moist **LOESS**					
4.0				DCP	☒ S-1	◆ 12	
7.0					☒ S-2		
12.0					☒ S-3		
12.5		Test pit completed at 12.5 feet bgs and backfilled with excavated material. Groundwater was not observed during excavation.	12.5				

Date Completed: 8/12/2015 Elevation: Final Depth: 12.5 feet bgs Latitude: 45.58483 Longitude: -122.45783

TP-16

DEPTH IN FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	DCP	COMMENTS
0.0		ROOT ZONE (4 inches)	0.3				
		Medium stiff brown orange SILT (ML) with gravel; non-plastic; fine, rounded gravel; moist **FILL**		DCP	☒ S-1	◆ 6	
3.5		Medium stiff brown orange SILT (ML); low plasticity; moist. **LOESS**	3.5		☒ S-2		
7.0		Brown orange silty GRAVEL (GM) with cobbles; non-plastic; coarse, subangular to angular gravel; subangular cobbles; moist **WEATHERED BASALT BEDROCK**	7.0		☒ S-3		
9.5		Test pit completed at 9.5 feet bgs and backfilled with excavated material. Refusal due to difficult excavation. Groundwater was not observed during excavation.	9.5				

Date Completed: 8/11/2015 Elevation: Final Depth: 9.5 feet bgs Latitude: 45.58580 Longitude: -122.45827

EXCAVATION METHOD: Backhoe with 24" Bucket LOGGED BY: T. Rikli

EXCAVATED BY: Dan J. Fischer Excavating, Inc.

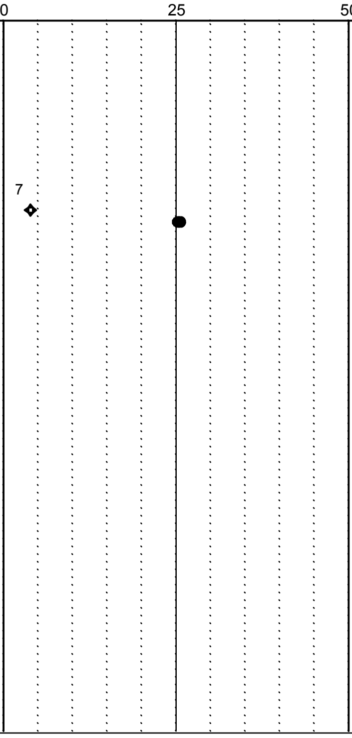
FIGURE A15

PBS	DAWSON'S RIDGE DEVELOPMENT NW MCINTOSH RD CAMAS, WASHINGTON	**TEST PITS**
	PBS PROJECT NUMBER: 73197.000	TEST PIT LOCATION: (See Site Plan)

DEPTH IN FEET	GRAPHIC LOG	MATERIAL DESCRIPTION NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	◆ DYNAMIC CONE PENETROMETER ☒ STATIC PENETROMETER ● MOISTURE CONTENT %	COMMENTS

TP-17

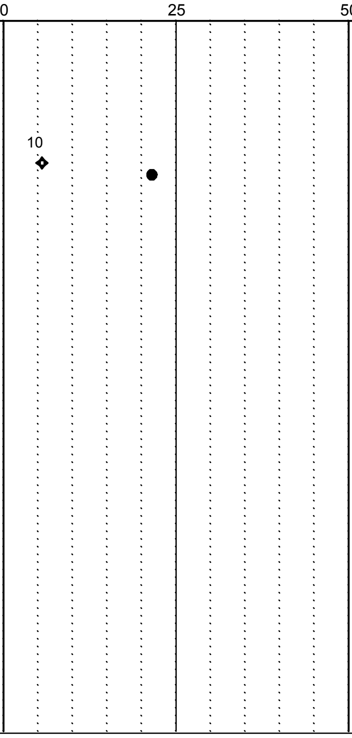
DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	COMMENTS
0.0	ROOT ZONE (3 inches)	0.3			
	Medium stiff brown gray SILT (ML); low plasticity; fine sand; moist **LOESS**				
4.0			P200 DCP	☒ S-1	P200 = 87%
8.0				☒ S-2	
10.5	Red poorly graded GRAVEL (GP-GM) with silt, sand, cobbles and boulders; non-plastic; fine to coarse sand; coarse, subangular to angular gravel; subangular cobbles; subangular boulders; moist	10.5		☒ S-3	
12.0	Test pit completed at 12 feet bgs and backfilled with excavated material. Groundwater was not observed during excavation.	12.0			



Date Completed: 8/12/2015 Elevation: Final Depth: 12 feet bgs Latitude: 45.58448 Longitude: -122.45702

TP-18

DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	COMMENTS
0.0	ROOT ZONE (3 inches)	0.3			
	Stiff brown orange SILT (ML) with sand; medium plasticity; fine sand; moist **LOESS**				LL = 40 PL = 26 PI = 14
3.0			ATT DCP	☒ S-1	
5.0				☒ S-2	
5.5	Test pit completed at 5.5 feet bgs and backfilled with excavated material. Terminated due to quipment break down. Groundwater was not observed during excavation.	5.5			



Date Completed: 8/13/2015 Elevation: Final Depth: 5.5 feet bgs Latitude: 45.58471 Longitude: -122.45571

EXCAVATION METHOD: Backhoe with 24" Bucket
EXCAVATED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: T. Rikli

FIGURE A17

PBS	DAWSON'S RIDGE DEVELOPMENT NW MCINTOSH RD CAMAS, WASHINGTON	**TEST PITS**
	PBS PROJECT NUMBER: 73197.000	TEST PIT LOCATION: (See Site Plan)

MATERIAL DESCRIPTION

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

◆ DYNAMIC CONE PENETROMETER
☒ STATIC PENETROMETER
● MOISTURE CONTENT %

TP-19

DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	◆ DCP / ● MOISTURE (0 – 25 – 50)	COMMENTS
0.0	ROOT ZONE (3 inches)	0.3				
	Very stiff brown orange SILT (ML) with sand; low plasticity; fine sand; moist **LOESS**					
2.0			DCP	☒ S-1	25 ◆	
4.0			P200	☒ S-2	●	P200 = 81%
6.0						
8.0				☒ S-3		
10.0	Test pit completed at 10 feet bgs and backfilled with excavated material. Groundwater was not observed during excavation.	10.0				
12.0						
14.0						

Date Completed: 8/11/2015 Elevation: Final Depth: 10 feet bgs Latitude: 45.58410 Longitude: -122.45470

TP-20

DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	◆ DCP / ● MOISTURE (0 – 25 – 50)	COMMENTS
0.0	Stiff brown SILT (ML) with sand; low plasticity; fine sand; moist **LOESS**					
2.0			DCP	☒ S-1	13 ◆	
4.0						
6.0				☒ S-2		
8.0						
10.0				☒ S-3		
	Test pit completed at 10.5 feet bgs and backfilled with excavated material. Groundwater was not observed during excavation.	10.5				
12.0						
14.0						

Date Completed: 8/13/2015 Elevation: Final Depth: 10.5 feet bgs Latitude: 45.58414 Longitude: -122.45583

EXCAVATION METHOD: Backhoe with 24" Bucket
EXCAVATED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: T. Rikli

FIGURE A19

PBS	DAWSON'S RIDGE DEVELOPMENT NW MCINTOSH RD CAMAS, WASHINGTON	**TEST PITS**
	PBS PROJECT NUMBER: 73197.000	TEST PIT LOCATION: (See Site Plan)

DYNAMIC CONE PENETROMETER ◆; STATIC PENETROMETER ☒; MOISTURE CONTENT % ●

NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.

TP-21

DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	DYNAMIC CONE PENETROMETER (0–25–50)	COMMENTS
0.0	ROOT ZONE (2 inches)					
	Stiff brown orange SILT (ML) with sand; low plasticity; fine sand; moist **LOESS**	0.2				
4.0			DCP	☒ S-1	◆ 13	
8.0				☒ S-2		
10.0	becomes gray					
12.0				☒ S-3		
	Test pit completed at 12.5 feet bgs and backfilled with excavated material. Groundwater was not observed during excavation.	12.5				

Date Completed: 8/13/2015 Elevation: Final Depth: 12.5 feet bgs Latitude: 45.58379 Longitude: -122.45461

TP-22

DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	DYNAMIC CONE PENETROMETER (0–25–50)	COMMENTS
0.0	ROOT ZONE (3 inches)					Encountered glass bottles
	Brown SILT (ML) with gravel; low plasticity; fine, rounded gravel; moist **FILL**	0.3				
	Medium stiff brown orange SILT (ML) with sand; low plasticity; fine sand; moist **LOESS**	2.5				
4.0			DCP	☒ S-1	◆ 8	
6.0				☒ S-2		
8.0	Red poorly graded GRAVEL (GP-GM) with silt, sand, cobbles and boulders; non-plastic; fine to coarse sand; coarse, subangular to angular gravel; subangular cobbles; subangular boulders; moist	8.0				
10.0				☒ S-3		
	Test pit completed at 11.5 feet bgs and backfilled with excavated material. Refusal due to difficult excavation. Groundwater was not observed during excavation.	10.5				

Date Completed: 8/12/2015 Elevation: Final Depth: 10.5 feet bgs Latitude: 45.58538 Longitude: -122.45798

EXCAVATION METHOD: Backhoe with 24" Bucket
EXCAVATED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: T. Rikli

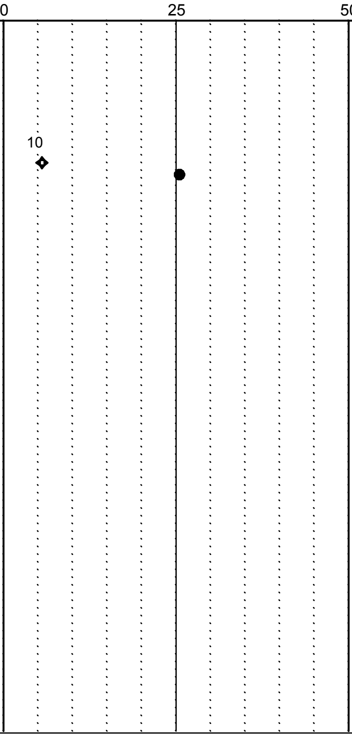
FIGURE A21

PBS	DAWSON'S RIDGE DEVELOPMENT NW MCINTOSH RD CAMAS, WASHINGTON	**TEST PITS**
	PBS PROJECT NUMBER: 73197.000	TEST PIT LOCATION: (See Site Plan)

DEPTH IN FEET	GRAPHIC LOG	MATERIAL DESCRIPTION NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	◆ DYNAMIC CONE PENETROMETER ☒ STATIC PENETROMETER ● MOISTURE CONTENT %	COMMENTS

TP-23

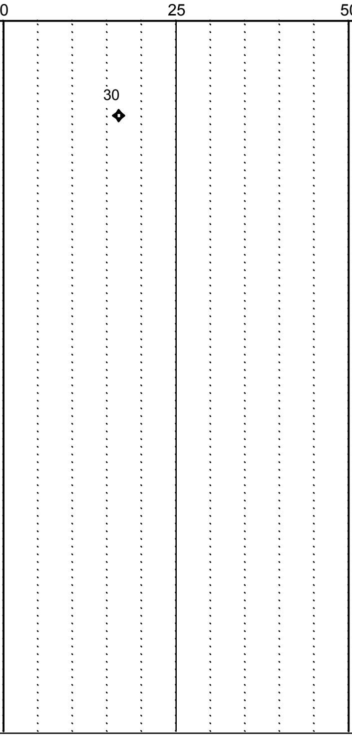
DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	COMMENTS
0.0	ROOT ZONE (2 inches)				
	Stiff brown CLAY (CL) with sand; high plasticity; fine sand; moist **LOESS**	0.2			
2.0			ATT DCP	☒ S-1	LL = 46 PL = 25 PI = 21
4.0					
6.0				☒ S-2	
8.0					
10.0				☒ S-3	
12.0	Test pit completed at 12 feet bgs and backfilled with excavated material. Groundwater was not observed during excavation.	12.0			
14.0					



Date Completed: 8/12/2015 Elevation: Final Depth: 12 feet bgs Latitude: 45.58360 Longitude: -122.45585

TP-24

DEPTH IN FEET	MATERIAL DESCRIPTION	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	COMMENTS
0.0	ROOT ZONE (2 inches)				
	Very stiff brown orange SILT (ML) with sand; low plasticity; fine sand; moist **LOESS**	0.2			
2.0			DCP	☒ S-1	
4.0					
6.0				☒ S-2	
8.0					
10.0				☒ S-3	
	Test pit completed at 10.5 feet bgs and backfilled with excavated material. Groundwater was not observed during excavation.	10.5			
12.0					
14.0					



Date Completed: 8/13/2015 Elevation: Final Depth: 10.5 feet bgs Latitude: 45.58327 Longitude: -122.45422

EXCAVATION METHOD: Backhoe with 24" Bucket LOGGED BY: T. Rikli
EXCAVATED BY: Dan J. Fischer Excavating, Inc.

FIGURE A23

DEPTH IN FEET	GRAPHIC LOG	MATERIAL DESCRIPTION NOTE: Lines representing the interface between soil/rock units of differing description are approximate only, inferred where between samples, and may indicate gradual transition.	DEPTH	TESTING	SAMPLE TYPE & SAMPLE ID	◆ DYNAMIC CONE PENETROMETER ☒ STATIC PENETROMETER ● MOISTURE CONTENT % 0 25 50	COMMENTS
TP-25							
0.0		ROOT ZONE (2 inches)	0.2				Encountered drain pipe
		Gray poorly graded GRAVEL (GP-GM) with silt and cobbles; non-plastic; coarse, angular gravel; angular cobbles; moist **FILL**	1.5				
2.0							
		Red poorly graded GRAVEL (GP-GM) with silt, sand, cobbles and boulders; non-plastic; fine to coarse sand; coarse, subangular to angular gravel; subangular cobbles; subangular boulders; moist **WEATHERED BASALT BEDROCK**			☒ S-1		
4.0							
6.0					☒ S-2		
		Test pit completed at 7 feet bgs and backfilled with excavated material. Refusal due to difficult excavation. Groundwater was not observed during excavation.	7.0				
8.0							
10.0							
12.0							
14.0							

Date Completed: 8/12/2015 Elevation: Final Depth: 7 feet bgs Latitude: 45.58200 Longitude: -122.45610

EXCAVATION METHOD: Backhoe with 24" Bucket
EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: T. Rikli

FIGURE A25

APPENDIX B

Laboratory Testing

APPENDIX B – LABORATORY TESTING

B1.0 GENERAL

The samples obtained during the field explorations were examined in the PBS laboratory. The physical characteristics of the samples were noted and the field classifications were modified where necessary. During the course of examination, representative samples were selected for further testing. The laboratory testing program adopted for this investigation included a variety of standard classification tests to provide data for the various engineering studies, which consisted of visual examination, moisture contents, and grain-size analyses. The classification tests yield certain index properties of the soils important to an evaluation of soil behavior. The testing procedures and results of the tests are presented in the following paragraphs. Unless noted otherwise, all test procedures followed applicable ASTM standards.

B2.0 CLASSIFICATION TESTS

B2.1 Visual Classification

The soils were classified in accordance with the Unified Soil Classification System with certain other terminology, such as the relative density or consistency of the soil deposits, in general accordance with engineering practice. In determining the soil type (i.e., gravel, sand, silt, or clay) the term which best described the major portion of the sample was used. Modifying terminology to further describe the samples is defined in Terminology Used to Describe Soil and Rock in Appendix A.

B2.2 Moisture (Water) Contents

Natural moisture content determinations were made on samples of the fine-grained soils (i.e., silts, clays, and silty sands). The natural moisture content is defined as the ratio of the weight of water to dry weight of soil, expressed as a percentage. The results of the moisture content determinations are presented on the test pit logs in Appendix A.

B2.3 Atterberg Limits

Atterberg limits were determined on selected samples for the purpose of classifying soils into various groups for correlation. The results of the Atterberg limits tests, which included liquid and plastic limits, are plotted on the Atterberg Limits Test Results, Figure B1, and on the test pit logs in Appendix A.

B2.4 Grain-Size Analysis

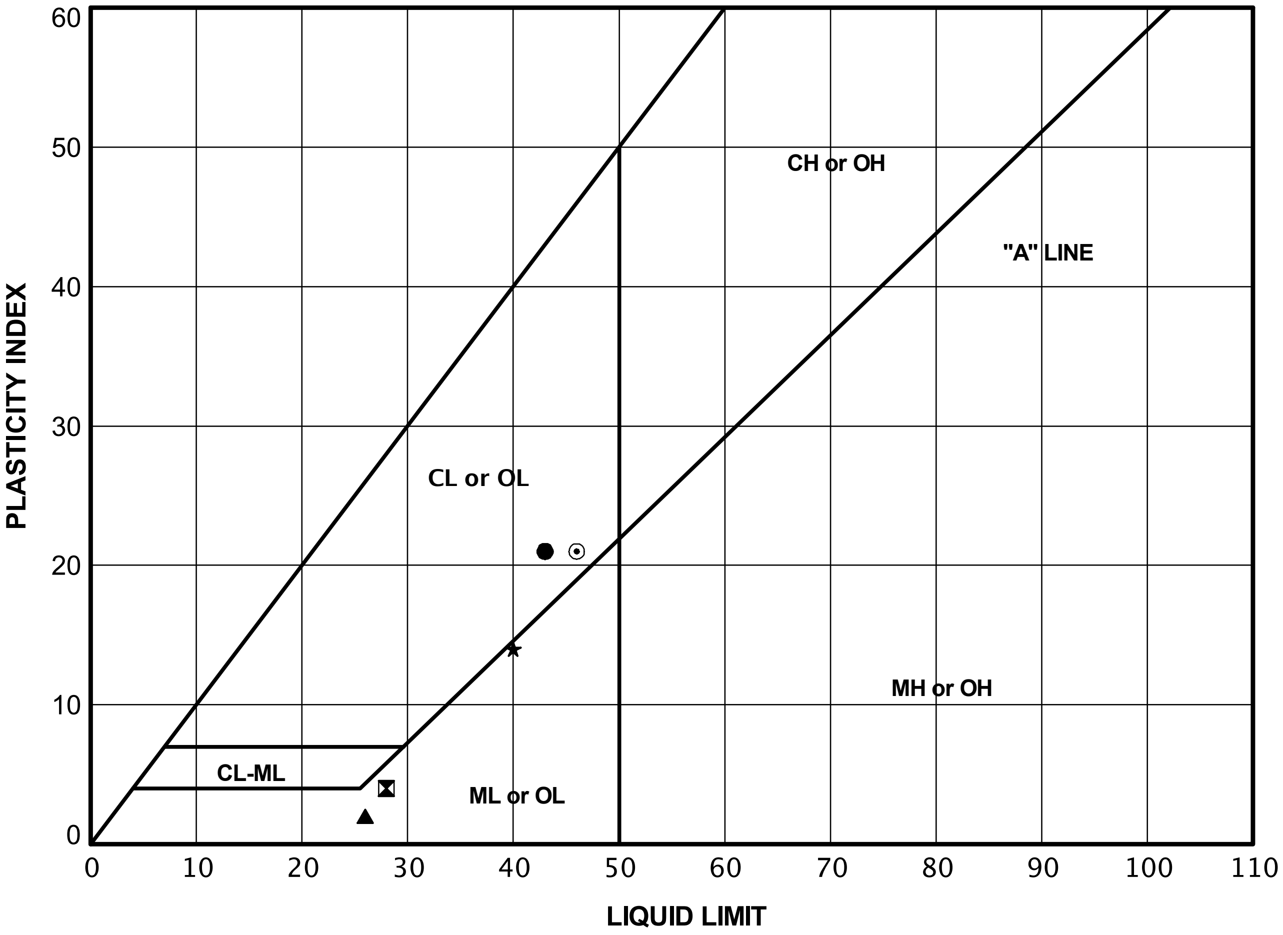
No. 200 washes (P200s) were completed on soil samples to determine the portion of soil passing the No. 200 Sieve (i.e., silt and clay). The sieve results are presented on the test pit logs in Appendix A.

ATTERBERG LIMITS TEST RESULTS

DAWSON'S RIDGE DEVELOPMENT
NW MCINTOSH RD
CAMAS, WASHINGTON

PBS PROJECT NUMBER:
73197.000

TEST METHOD: ASTM D4318



KEY	EXPLORATION NUMBER	SAMPLE NUMBER	SAMPLE DEPTH (FEET)	NATURAL MOISTURE CONTENT (PERCENT)	PERCENT PASSING NO. 40 SIEVE (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	TP-1	S-1	3.0	22	NA	43	22	21
☒	TP-7	S-2	6.0	24	NA	28	24	4
▲	TP-13	S-2	6.0	26	NA	26	24	2
★	TP-18	S-1	3.0	21	NA	40	26	14
⊙	TP-23	S-1	3.0	26	NA	46	25	21

FIGURE B1